SEATTLE REGIONAL MEETING—NOVEMBER 5-6, 1957

AUGUST 1957
No. 2
VOL. 29

JOURNAL
OF THE
AMERICAN CONCRETE INSTITUTE

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Coming next month in the JOURNAL

Introducing a series of reports on the ACI Building Code, FRANK KEREKES offers a "Review of Changes in the ACI Building Code Requirements for Reinforced Concrete." The former chairman of Committee 318 summarizes changes, relates code content to teaching and to practice.

"Explanatory Notes on Appendix to 'Building Code Requirements for Reinforced Concrete (ACI 318-56)' " have been prepared by COMMITTEE 318 in response to requests for elaboration of certain sections.

"Failures of Concrete Structures" by JACOB FELD surveys concrete failures of the last half century in the United States, touching on legal aspects from the Code of Hammurabi through English common law with some observations on present code requirements.

BRYANT MATHER's paper "Laboratory Tests of Portland Blast-Furnace Slag Cements" in the September issue gives a detailed report of tests made on samples of portland blast-furnace slag cements, and of blast-furnace slags and portland-cement clinkers used in their manufacture.

F. T. MAVIS and M. J. GREAVES have compiled extensive research data in their report "Destructive Impulse Loading of Reinforced Concrete Beams." Typical tests are detailed in the paper and dynamic behavior phases are identified and interpreted.

Manuscripts of papers, discussions, and reports should be sent in triplicate to:

Secretary, Technical Activities Committee
AMERICAN CONCRETE INSTITUTE
P. O. BOX 4754, REDFORD STATION, DETROIT 19, MICHIGAN

Proposed

Test Procedure to Determine Relative Bond Value of Reinforcing Bars*

Reported by ACI Committee 208

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SYNOPSIS

This test procedure provides a uniform basis for comparison of bond qualities of different reinforcing bars. The recommended test method uses companion beam type specimens, cast horizontally in inverted and normal positions so that the effects of water gain and settlement of concrete are included in the evaluation. The concrete surrounding the bars is in tension as bars are ordinarily used, which makes the results more acceptable to some users. Minimum criteria for acceptance are not included since the purpose is merely to establish relative bond values for the different bars under consideration.

The test may be used to evaluate the effectiveness of deformed bars having characteristics other than those described in "Tentative Specifications for Minimum Requirements for the Deformations of Deformed Steel Bars for Concrete Reinforcement," ASTM A 305-56T. For construction under the ACI Building Code, an evaluation of such bars is necessary since the code definition for a deformed bar is one which meets the requirements of ASTM A 305. Manufacturers of bars meeting the requirements of ASTM A 305 may use the test procedure for product improvement and to evaluate modifications in the design of deformations.

BEAM SPECIMENS

1. There shall be 18 beam specimens, the details of which are shown on the accompanying Drawings 1 and 2.†

2. The nine A, B, and C specimens (three for each length of embedment) shall be cast erect as shown. The nine AA, BB, and CC specimens shall be cast inverted.

*Title No. 54-6 is a part of copyrighted JOURNAL OF THE AMERICAN CONCRETE INSTITUTE, V. 29, No. 2, Aug. 1957, *Proceedings* V. 54. Separate prints are available at 60 cents each. **Discussion** (copies in **triplicate**) should reach the Institute not later than Nov. 1, 1957. Address P. O. Box 4754, Redford Station, Detroit 19, Mich.

This report was submitted to letter ballot of the committee which consists of 9 members, with 9 voting affirmatively. It is released by the Standards Committee for publication and discussion with view to its consideration for adoption as an Institute Standard at the 54th Annual Convention, Chicago, Feb. 24-27, 1958.

†The 11 drawings, pp. 94-104, which constitute an important part of the test procedure are available in blueprints (22 x 35 in.) *in complete sets only* at $36.00 per set from ACI headquarters.

CONCRETE

3. The concrete shall have a slump of 5 to 6 in. in the standard slump test, and an ultimate strength of 3500 psi (plus or minus 300 psi) at age of 28 days. Preliminary tests shall be made to determine the proportions of the prescribed materials required to develop this ultimate strength. The cement shall be equal parts by weight of four standard brands of Type I portland cement. The fine aggregate shall be natural sand well graded from 0 to No. 4 sieve. The coarse aggregate shall be gravel, all of which shall pass either a sieve having $\frac{3}{4}$ in. square openings or a screen with 1 in. diameter openings. All materials shall conform to ASTM standards.

BARS

4. The deformed or plain bars to be tested for bond shall be nominally $\frac{7}{8}$ in. in diameter. The weight shall be measured and reported in lb per ft to the nearest 0.01 lb. In the interest of uniformity and simplicity, values for tensile stress, yield point, bond stress, and modulus of elasticity will be calculated on the basis of the cross-sectional area and the perimeter listed in handbooks for a plain round bar of the same nominal diameter. Specifically the cross-sectional area of a $\frac{7}{8}$-in. deformed or plain bar (meeting ASTM requirements for weight tolerances) will be taken as 0.60 sq in. and the perimeter as 2.75 in.

5. The pull at yield point and the relation between pull on bar and elongation shall be determined from several typical samples of each design of bar. This relation will serve to indicate the pull on bar at various stages of the test from the strain measurements to be made at the sides of the bar at notches cast in the beam specimens.

6. The bars submitted for test shall be representative of the product of the mill which submits them, and shall not be given any treatment to produce an abnormal surface. The surface of all bars, however, shall be thoroughly cleaned with naphtha or carbon tetrachloride just before insertion in mold to remove all oil or grease and any other foreign matter. If, in any case, there is a question as to whether the bars submitted are truly representative, new samples shall be selected for the tests by an experienced disinterested inspector.

MOLDS AND ASSEMBLIES

7. The mold shall be of metal, and shall be coated to prevent adhesion to the concrete and sealed to prevent leakage. The mold assembly is shown in Drawing 4 and the exterior parts of mold are detailed in Drawing 5. Two notches 6 in. long and 3 in. high shall be cast in all beam specimens, as shown in Drawings 1, 3, and 4, and the insert for these notches shall be as detailed in Drawing 6.

8. Details of the main reinforcing bar and stirrup assembly ready for installing in the mold are shown in Drawings 10 and 11.

9. The $\frac{7}{8}$-in. bar shall be so placed in the mold that the parting line of its deformation pattern (that is, the parting line of upper and lower halves of bar as rolled) shall come at the sides (see detail, Drawing 4). The 3-in. extensions of the bar beyond the center lines of the reactions shall be covered with a rubber sleeve so as to insure no bond. The stirrups shall be $\frac{1}{2}$-in. round deformed bars, furnished by the maker of the $\frac{7}{8}$-in. bars to be tested, and shall be welded into loops as shown in Drawing 11. All steel reinforcement shall be rigidly held in place in the mold (see Drawing 4).

10. Within the length of each notch, two yokes, similar to those shown in Drawing 3 and detailed in Drawing 7 shall be clamped into carefully drilled contact points on 5-in. centers on each side of the bar. These points shall be so located (by means of the jig detailed in Drawing 10) as to make the two yokes as nearly parallel as is practicable. The pull or the fiber stress in the bar shall be determined from the average of the change in distance between the two yokes.

CASTING, REMOVAL OF FORMS, CURING

11. The three identical specimens representing one length of embedment of a given bar shall be made on different days to average minor differences in the mix and other variables. The concrete shall be deposited in two layers and each layer shall be sliced and rodded to insure complete contact with the steel reinforcement. Immediately after placing, the concrete should be struck off level with the top edge of the mold with a minimum of disturbance. The specimens must not be exposed to any vibrations or other disturbances during the first 24 hr after casting.

12. At least three standard 6 x 12-in. cylinders for compression tests shall be made from the various batches of concrete mixed for each beam, one cylinder from any one batch.

13. After preliminary curing in the mold for 5 days and removal of molds as indicated below, the beam specimens and cylinders shall be stored in a standard fog room at a temperature of 65 to 75 F and tested moist at age of 28 days. While in the molds the beams and cylinders shall be protected against loss of moisture from the concrete.

14. To avoid damaging the beam specimens the sides and ends of the molds and the tapered inserts shall not be removed earlier than 5 days after placing the concrete. Care shall be exercised to avoid damaging the green specimens during removal to moist room. To this end it is recommended that the beams be placed on stiff wood planks or equivalent supports during trucking operations. The pieces forming the notches in the beam are to be withdrawn at the end of the 5-day period in the manner shown in Drawing 6.

TESTING

15. The sides of the specimen shall be whitewashed about 2 hr before test and allowed to dry so that the sequence and development of cracks may be easily marked by pencil lines which will be visible on any photographs taken after test.

16. Loads shall be applied through rollers or rockers. The beams shall rest on rocker or roller supports to avoid restraint to elongation of bottom fibers of beam as load is applied. The apparatus for supporting and applying load to beam specimen is shown in Drawing 3 and detailed in Drawing 9.

17. A reference steel angle clip shall be screwed to the concrete of the beam sides as near as is convenient to the notch and on the reaction side of the load as shown in Drawing 3 and detailed in Drawing 7. Between this angle and the nearer extensometer yoke the slip of bar immediately under the load shall be measured. This slip will be designated the slip at "loaded" end of bar.

18. The slip of bar at end of beam shall be measured in the manner shown in Drawing 3 with the gage detailed in Drawing 7. This slip will be designated as the slip at "free" end of bar.

19. The deflection at center of the beam shall be measured in the manner shown in Drawing 3 with the apparatus detailed in Drawing 8.

20. The rate of load application during the beam tests shall not be greater than 2000 lb per min nor greater than is consistent with securing reliable test data. Readings may be secured for continuous application of load if available manpower permits. With a shortage of observers it may be preferable either to stop the machine for securing readings at each increment of load or not to stop the machine but to read only half the gages as a given load is passed; reading the other half of the gages at the next increment. When readings are staggered in this manner the indications of corresponding pairs of gages on opposite sides of the beam should be noted simultaneously as in the case of gages indicating slip and strain at a given notch.

21. Load shall be applied to failure, and the report shall state whether failure of the beam was due to failure in bond, diagonal tension, failure of bottom bar in tension or of top concrete in compression, or by splitting of concrete as a result of failure in bond. Cracks due to settlement of concrete will normally occur in the top surface of the beam specimens AA, BB, and CC cast inverted. These should be noted and diagrammed for later reference.

22. Readings of slip at both free and loaded ends of the bar, of pull in the bar, and of deflection of the beam at center shall be taken in such number (preferably not less than 15) as to give smooth curves in graphical representations of the test data.

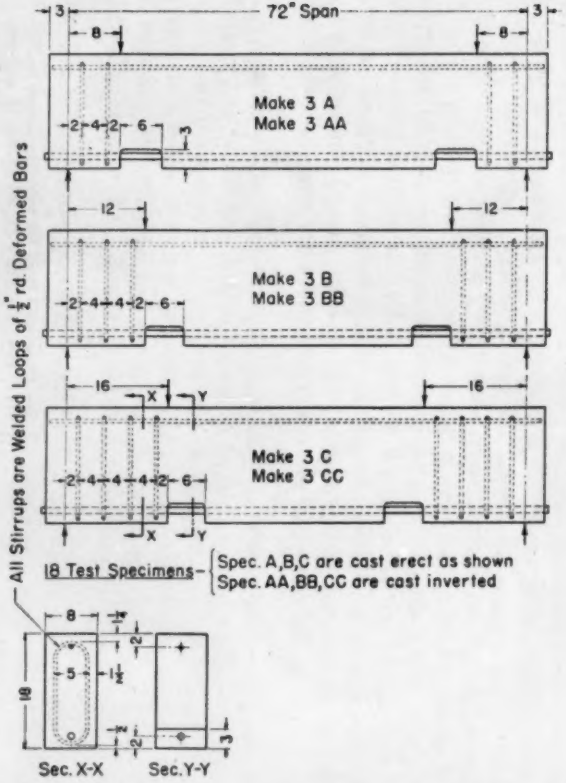
REPORT

23. Curves showing slip of bar shall have the amounts of slip as abscissae, and the beam loads, pulls or fiber stresses in bar, unit bond stresses, and values of u/f_c' as ordinates. The unit bond stress u is the value obtained by dividing the indicated pull on the bar in pounds (from strain measurements) by the area in bond. The area in bond is obtained by multiplying the nominal perimeter of 2.75 in. for the $\frac{7}{8}$-in. diameter bar (see Paragraph 4) by the effective length of embedment (8, 12, or 16 in.). Load-deflection curves shall show the amount of deflection as abscissae and beam loads as ordinates.

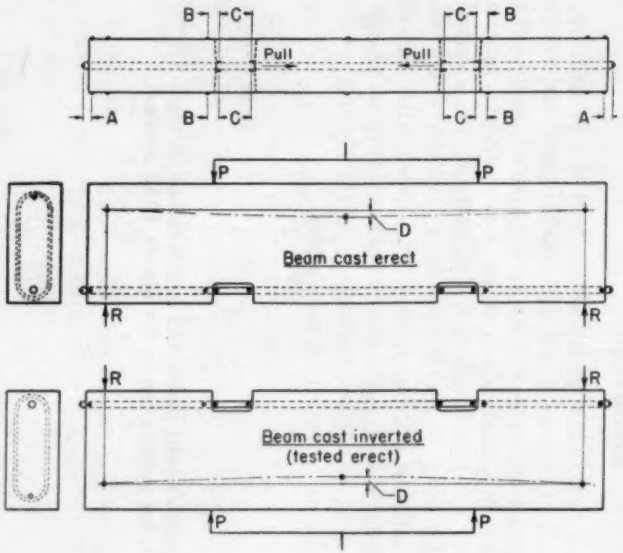
24. The report shall contain a detailed description of the testing setup, and of all devices and measuring instruments employed.

Discussion of this report should reach ACI headquarters in triplicate by Nov. 1, 1957, for publication in the March 1958 JOURNAL.

[See Drawings 1-11 on the following pages: pp. 94-104. Blueprints (22 x 35 in.) of the 11 drawings are available in complete sets only at $36.00 per set from ACI headquarters.]



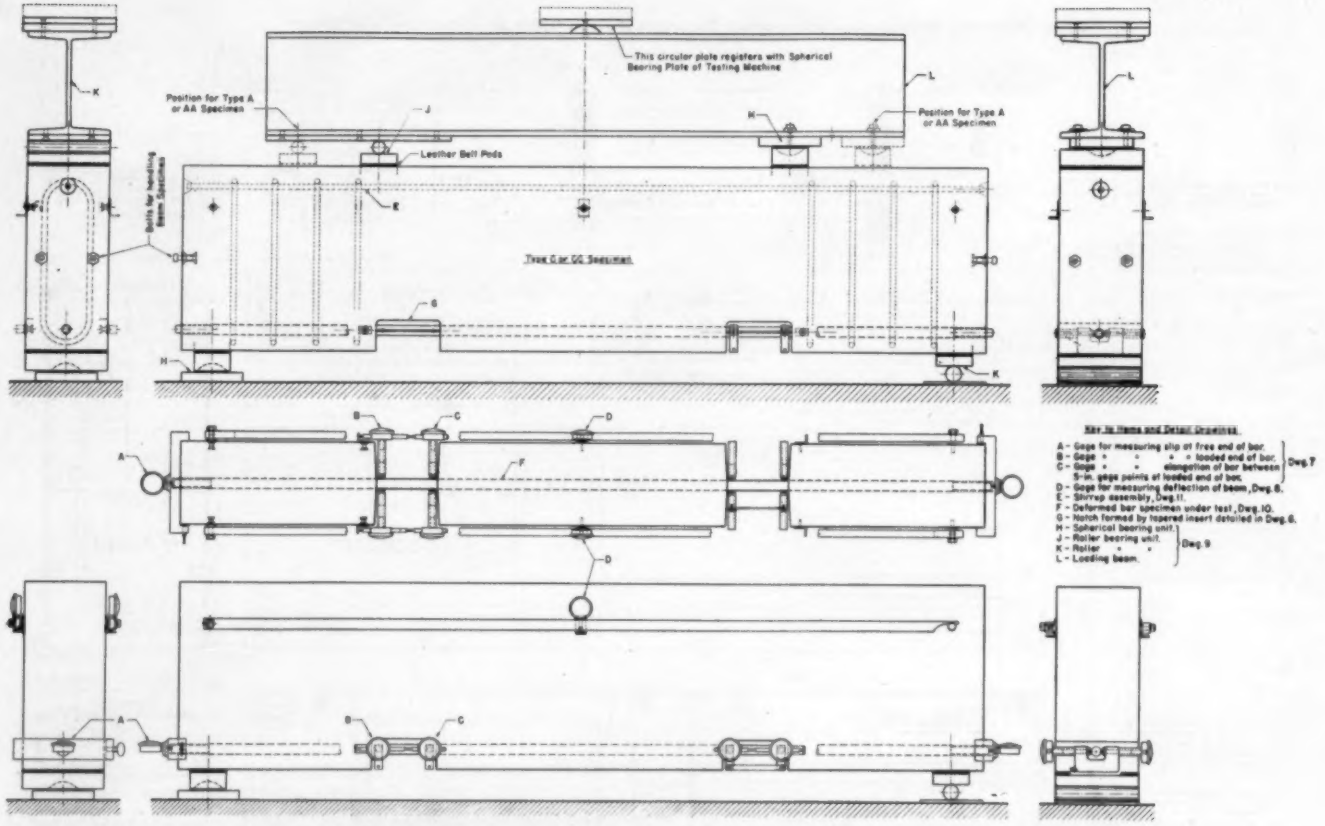
Dwg. 1—Beam specimens for determining the "relative bond value" of reinforcing bars with various types of deformations



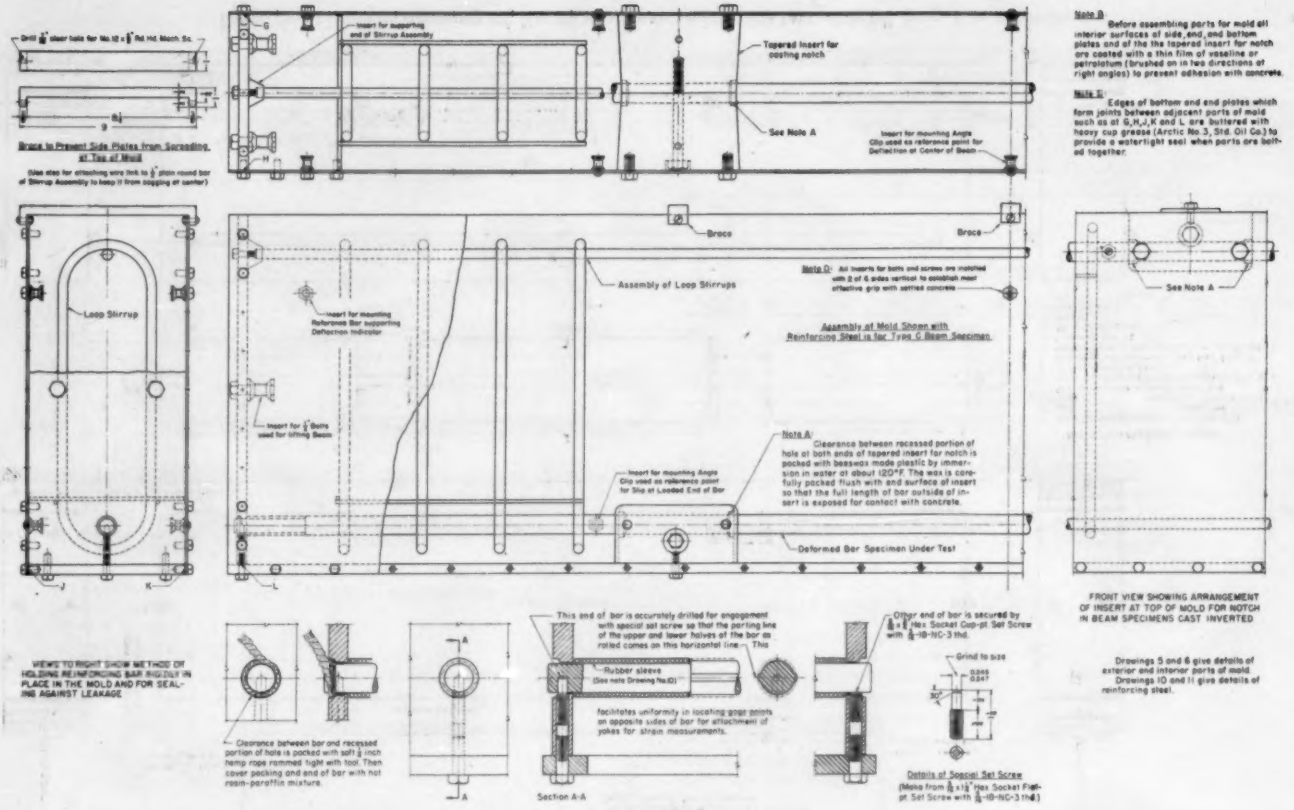
Reference Points

A - Slip at free end of bar
B - Slip at loaded end of bar
C - Elongation of bar between 5" gage points at loaded end
D - Deflection at center of beam near neutral surface
P - Applied load
R - Reaction to applied load

Dwg. 2—Diagrams showing reference points for measurements on beam specimens

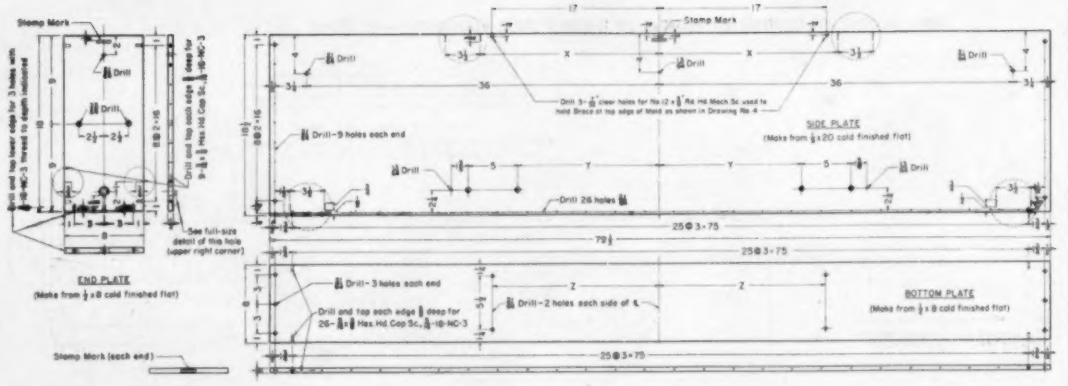

This circular plate registers with Spherical Bearing Plate of Testing Machine
Position for Type A or AA Specimen
Position for Type A or AA Specimen
Leather Belt Pads
Belts for handling Beam Specimen
Type C or CC Specimen
Key to Items and Detail Drawings
A - Gage for measuring slip at free end of bar.
B - Gage " " " " loaded end of bar.
C - Gage " " elongation of bar between 5-in. gage points at loaded end of bar.
Dwg.7
D - Gage for measuring deflection of beam, Dwg.8.
E - Stirrup assembly, Dwg.11.
F - Deformed bar specimen under test, Dwg.10.
G - Notch formed by tapered insert detailed in Dwg.6.
H - Spherical bearing unit.
J - Roller bearing unit.
K - Roller " "
L - Loading beam.
Dwg.9

Dwg. 3—General drawing of beam specimen ready for test with all gages in position

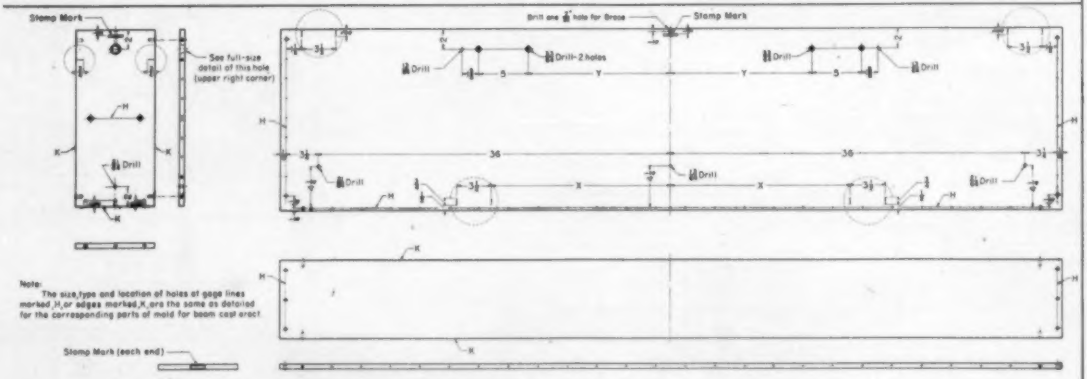

Insert for supporting end of Stirrup Assembly
Tapered insert for casting notch
See Note A
Insert for mounting Angle Clip used as reference point for Deflection at Center of Beam
Brace to Prevent Side Plates from Spreading at Top of Mold
Loop Stirrup
Brace
Brace
Assembly of Loop Stirrups
Assembly of Mold Shown with Reinforcing Steel is for Type C Beam Specimen
Insert for mounting Reference Bar supporting Deflection Indicator
Insert for 1/4 Bolts used for lifting Beam
Insert for mounting Angle Clip used as reference point for Slip at Loaded End of Bar
Deformed Bar Specimen Under Test
Note A: Clearance between recessed portion of hole at both ends of tapered insert for notch is packed with beeswax made plastic by immersion in water at about 120°F. The wax is carefully packed flush with and surface of insert so that the full length of bar outside of insert is exposed for contact with concrete.
Note B: Before assembling parts for mold all interior surfaces of side, end, and bottom plates and of the the tapered insert for notch are coated with a thin film of vaseline or petrolatum (brushed on in two directions at right angles) to prevent adhesion with concrete.
Note C: Edges of bottom and end plates which form joints between adjacent parts of mold such as at G, H, J, K and L are buttered with heavy cup grease (Arctic No. 3, Std. Oil Co.) to provide a watertight seal when parts are bolted together.
Note D: All inserts for bolts and screws are installed with 2 of 6 sides vertical to establish most effective grip with settled concrete.
See Note A
FRONT VIEW SHOWING ARRANGEMENT OF INSERT AT TOP OF MOLD FOR NOTCH IN BEAM SPECIMENS CAST INVERTED
VIEWS TO RIGHT SHOW METHOD OF HOLDING REINFORCING BAR RIGIDLY IN PLACE IN THE MOLD AND FOR SEALING AGAINST LEAKAGE
Clearance between bar and recessed portion of hole is packed with soft 1/2 inch hemp rope rammed tight with tool. Then cover packing and end of bar with hot rosin-paraffin mixture.
Section A-A
This end of bar is accurately drilled for engagement with special set screw so that the parting line of the upper and lower halves of the bar as noted comes on this horizontal line. This
Rubber sleeve (See note Drawing No. 10)
facilitates uniformity in locating gage points on opposite sides of bar for attachment of yokes for strain measurements.
Other end of bar is secured by 1/4-20 Hex Socket Cup-pt. Set Screw with 1/4-10-NC-3 thd.
Grind to size
Details of Special Set Screw (Make from 1/4 x 1 1/2" Hex Socket Flat-pt. Set Screw with 1/4-10-NC-3 thd.)

Dwg. 4—Assembly of mold with reinforcing steel ready to receive concrete

Drawings 5 and 6 give details of exterior and interior parts of mold. Drawings 10 and 11 give details of reinforcing steel.


Stamp Mark
Drill
END PLATE
(Make from ½ x 8 cold finished flat)
See full-size detail of this hole (upper right corner)
Stamp Mark (each end)
Drill 3-⅞" clear holes for No.12 x 1½" Rd Hd Mach Sc used to hold Brace at top edge of Mold as shown in Drawing No. 4
SIDE PLATE
(Make from ½ x 20 cold finished flat)
Drill-9 holes each end
Drill 26 holes
25@3=75
79½
Drill-3 holes each end
Drill and tap each edge ½ deep for 26-⅝ x 1½ Hex. Hd. Cap Sc., ⅝-18-NC-3
Drill-2 holes each side of ℄
BOTTOM PLATE
(Make from ½ x 8 cold finished flat)

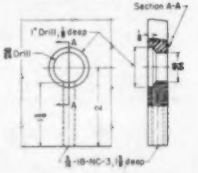
EXTERIOR PARTS OF MOLD FOR BEAMS CAST ERECT (Specimens A,B,C)


Stamp Mark
Drill one ⅞" hole for Brace
See full-size detail of this hole (upper right corner)
Drill-2 holes
Stamp Mark (each end)

Note:
The size, type and location of holes at gage lines marked H, or edges marked K, are the same as detailed for the corresponding parts of mold for beam cast erect.

EXTERIOR PARTS OF MOLD FOR BEAMS CAST INVERTED (Specimens AA,BB,CC)

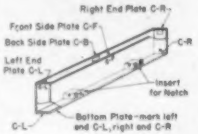
Detail of Recessed Hole in End Plate



Values of Dimensions X, Y, Z for Beam Molds

Specimen	Casting Position	Length of Embedment	X	Y	Z
A	Erect	8	26½	22½	25
AA	Inverted	8	"	"	"
B	Erect	12	22½	18½	21
BB	Inverted	12	"	"	"
C	Erect	16	18½	14½	17
CC	Inverted	16	"	"	"

Marking System for Identifying Parts of Mold for a Given Beam Specimen



The above diagram illustrates marking of mold parts for Type C specimens. The first letter indicates the type of beam and the second letter the location of part in diagram of mold assembly. These marks are stamped into outer face of each part with steel letters ½" high. The 6 types of specimens (one complete mold for each type) are indicated in the table of dimensions above.

Cast-in Marks on Beam Specimens to Assist in Mounting Apparatus For Test

The pair of short dash lines detailed within circled areas of the side plates represent the location and length of V-shaped grooves milled into the inner face of each front and back plate. These grooves leave small projecting lines on the surface of the concrete and mark the correct position of bearing plates at load and reaction points during test of spec.

Grooves are also milled into the inner face of all end plates to mark position of yoke of gage for strip measurements at free ends of spec.

Dwg. 5—Details of exterior parts of mold for beam specimens cast erect and inverted

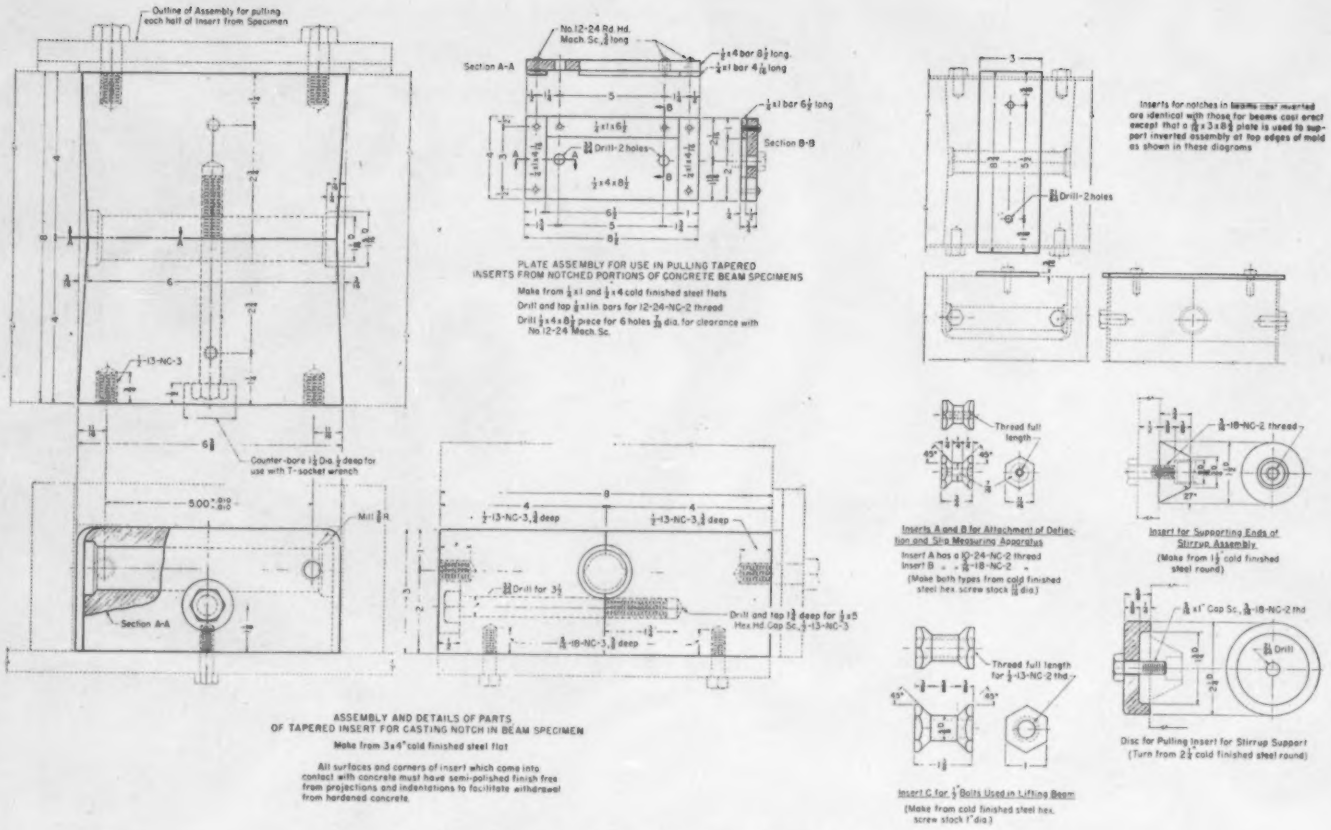

Outline of Assembly for pulling each half of insert from Specimen
Section A-A
Section B-B
Drill-2 holes
Counter-bore 1½ Dia. ½ deep for use with T-socket wrench
Mill ⅛ R
Drill and tap 1½ deep for ½x5 Hex Hd. Cap Sc.-13-NC-3
Thread full length
Thread full length for ½-13-NC-2 thd
Drill
Disc for Pulling Insert for Stirrup Support (Turn from 2½ cold finished steel round)
Insert for Supporting Ends of Stirrup Assembly (Make from 1½ cold finished steel round)
Inserts for notches in beams cast inverted are identical with those for beams cast erect except that a ⅛x3x8½ plate is used to support inverted assembly at top edges of mold as shown in these diagrams

PLATE ASSEMBLY FOR USE IN PULLING TAPERED INSERTS FROM NOTCHED PORTIONS OF CONCRETE BEAM SPECIMENS

Make from ½x1 and ½x4 cold finished steel flats

Drill and tap ½x1 in. bars for 12-24-NC-2 thread

Drill ½x4x8½ piece for 6 holes ⅝ dia. for clearance with No. 12-24 Mach. Sc.

ASSEMBLY AND DETAILS OF PARTS OF TAPERED INSERT FOR CASTING NOTCH IN BEAM SPECIMEN

Make from 3x6" cold finished steel flat

All surfaces and corners of insert which come into contact with concrete must have semi-polished finish free from projections and indentations to facilitate withdrawal from hardened concrete.

Inserts A and B for Attachment of Deflection and Slip Measuring Apparatus

Insert A has a 10-24-NC-2 thread

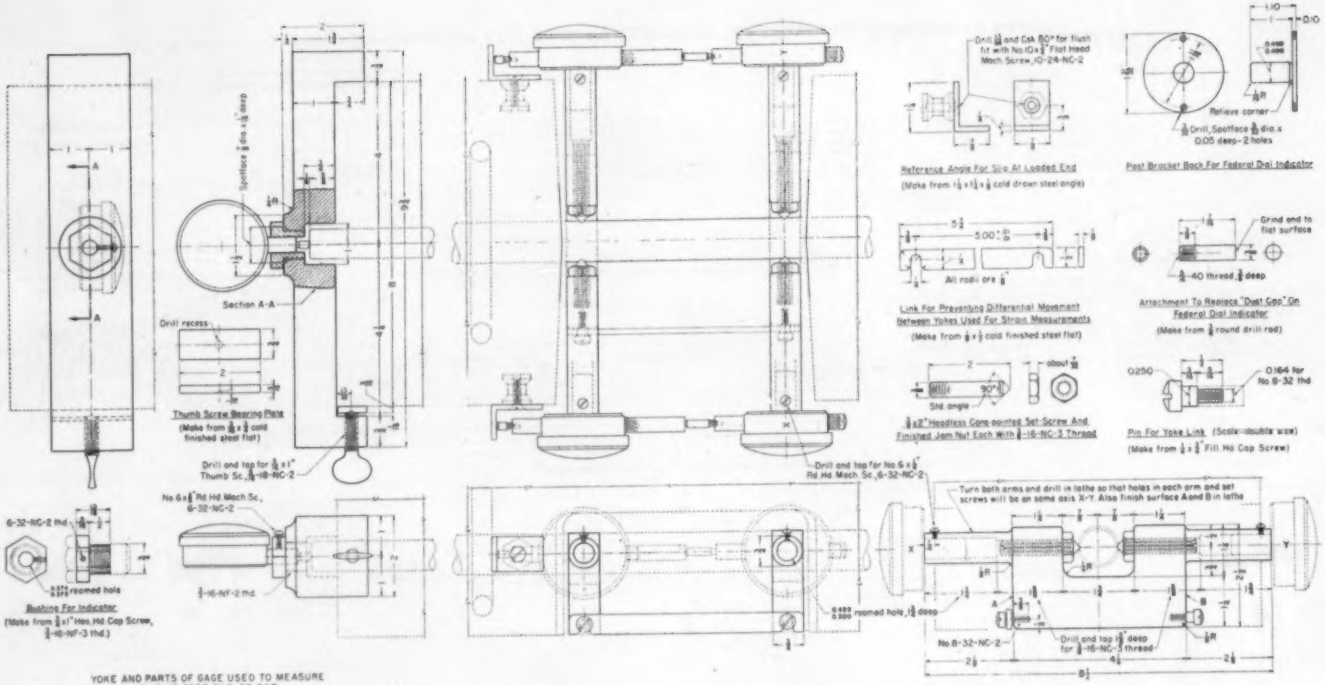
Insert B " " ⅜-18-NC-2 "

(Make both types from cold finished steel hex. screw stock ⅝ dia.)

Insert C for ½ Bolts Used in Lifting Beam

(Make from cold finished steel hex. screw stock 1" dia.)

Dwg. 6—Assembly and details of inserts at interior of beam mold


Section A-A
Drill recess
Spotface ⅜ dia. x ⅛ deep
Thumb Screw Bearing Plate
(Make from ⅛ x ½ cold finished steel flat)
Drill and tap for ⅜ x 1" Thumb Sc., ⅜-16-NC-2
No 6 x ⅜ Rd Hd Mach Sc., 6-32-NC-2
6-32-NC-2 thd
0.377 0.375 reamed hole
Bushing For Indicator
(Make from ⅜ x 1" Hex. Hd Cap Screw, ⅜-16-NF-3 thd.)
⅜-16-NF-2 thd
Drill and tap for No 6 x ⅜ Rd Hd Mach. Sc., 6-32-NC-2
0.377 0.375 reamed hole, ⅜ deep
Drill ⅜ and Csk 80° for flush fit with No.10 x ⅜ Flat Head Mach. Screw, 10-24-NC-2
Reference Angle For Slip At Loaded End
(Make from 1½ x 1½ x ⅛ cold drawn steel angle)
Relieve corner
⅜ Drill, Spotface ⅝ dia. x 0.05 deep - 2 holes
Post Bracket Back For Federal Dial Indicator
5.00" ±0.01
All radii are ⅛"
Link For Preventing Differential Movement Between Yokes Used For Strain Measurements
(Make from ⅜ x ½ cold finished steel flat)
Grind end to flat surface
⅜-40 thread ⅜ deep
Attachment To Replace "Dust Cap" On Federal Dial Indicator
(Make from ⅜ round drill rod)
Std. angle
⅜ x 2" Headless Cone-pointed Set-Screw And Finished Jam Nut Each With ⅜-16-NC-3 Thread
0.250
0.164 for No 6-32 thd
Pin For Yoke Link (Scale—double size)
(Make from ½ x ⅜" Fill Hd Cap Screw)
Turn both arms and drill in lathe so that holes in each arm and set screws will be on same axis X-Y. Also finish surface A and B in lathe
No 6-32-NC-2
Drill and tap ⅞ deep for ⅜-16-NC-3 thread

YOKE AND PARTS OF GAGE USED TO MEASURE SLIP AT FREE END OF BAR

(Make yoke from 2" square straight drawn aluminum bar, alloy 17 ST)

YOKES AND PARTS FOR GAGE USED TO MEASURE ELONGATION (PULL) AND SLIP AT LOADED END OF BAR

Only one link is to be used between each pair of yokes but link pins are to be provided on both shoulders of each yoke so that link can be inserted at either side of notch. The yokes in each pair are identical.

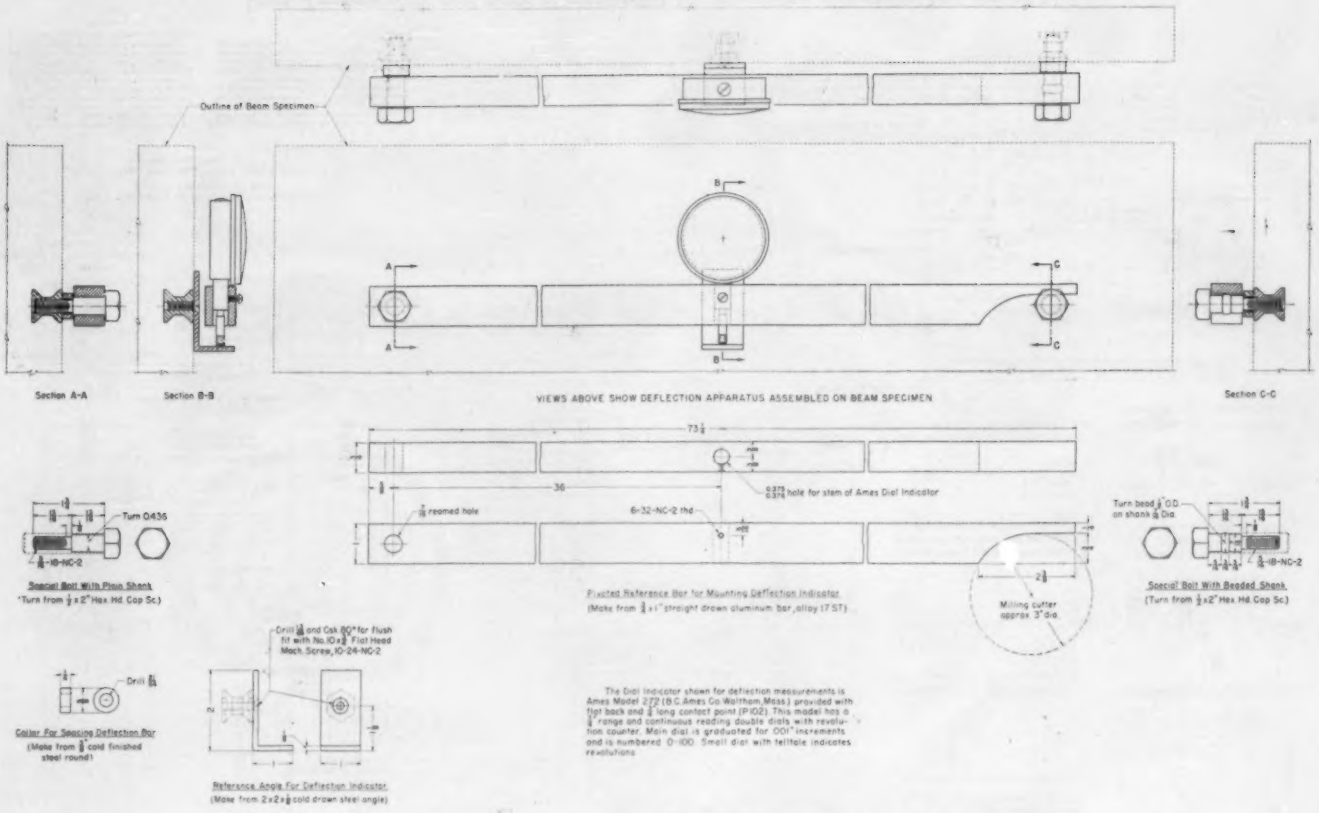
Dial indicators shown for slip and elongation measurements are Federal Model G-21 (Federal Products Corp. Providence, R.I.). They are to have continuous reading double dials with revolution counter and a range of .2000". Main dial is graduated for .0001" increments and is numbered 0-100. Small dial with letters indicates revolutions.

Indicators for slip at free end of bar have flat backs and ⅛" long contact points. Indicators for slip at loaded end have post bracket backs, ⅛" long contact points and an attachment replacing usual "dust cap" indicators for elongation measurements have post bracket backs and points ⅜ long.

Extra contact points ⅛ x ⅛ x ½ and ⅜ long (Two for each length) should be available. All points are of the regular type with ⅜ dia. and ⅛ contact rod.

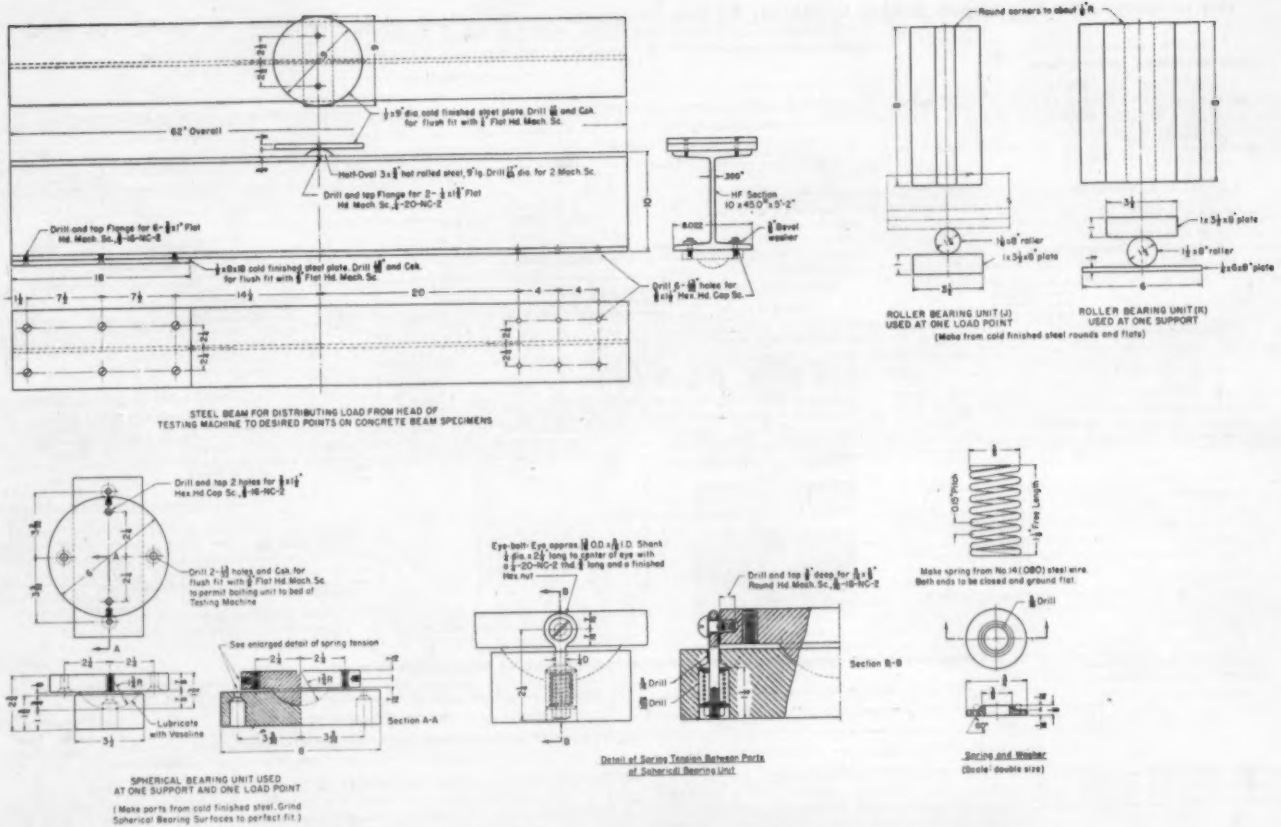
Indicators should be checked and used over ranges of greatest accuracy, particularly those for elongation measurements. At least three indicators, one with flat back and two with post backs, should be available as spares.

Dwg. 7—Assembly and parts of apparatus for measuring elongation and slip of bar

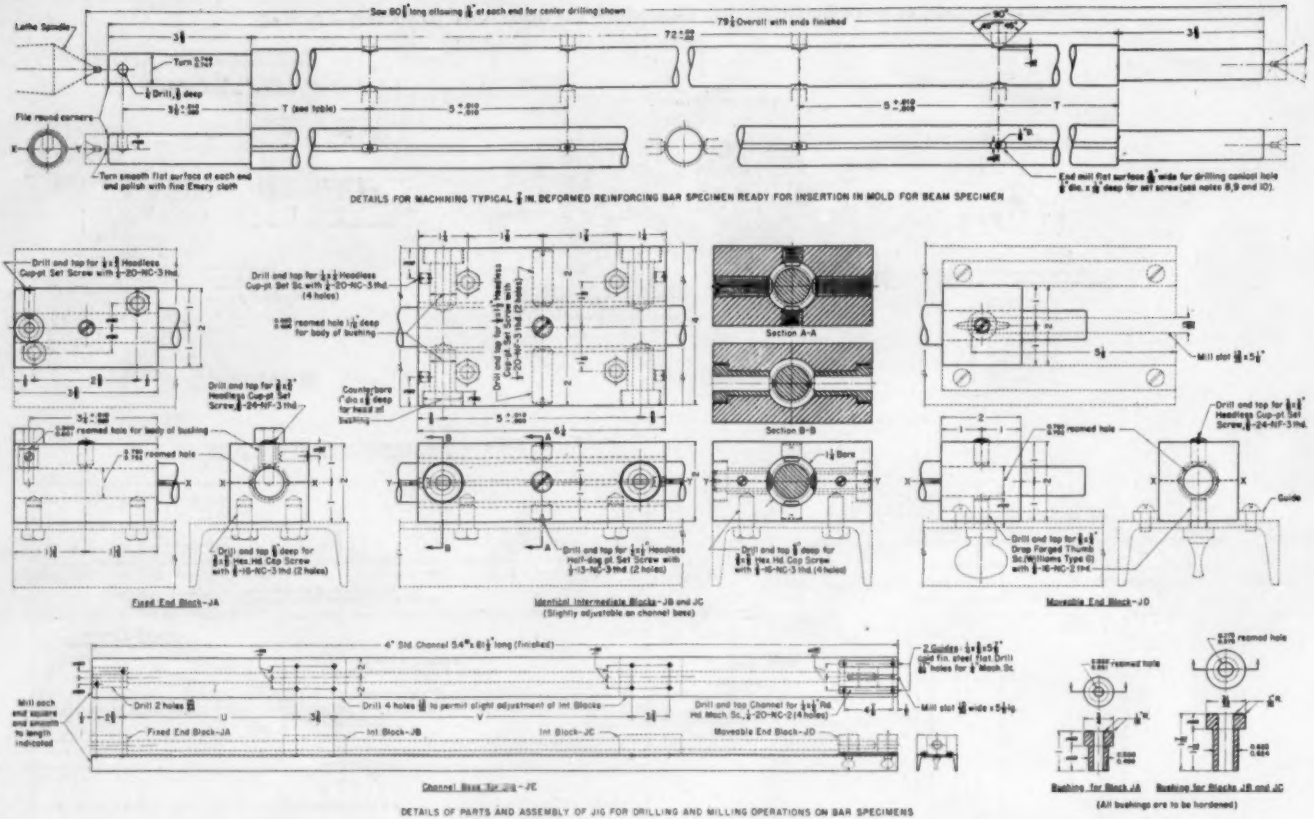

Outline of Beam Specimen
Section A-A
Section B-B
VIEWS ABOVE SHOW DEFLECTION APPARATUS ASSEMBLED ON BEAM SPECIMEN
Section C-C
Special Bolt With Plain Shank
(Turn from ½ x 2" Hex Hd. Cap Sc.)
Collar For Spacing Deflection Bar
(Make from ⅜ cold finished steel round)
Reference Angle For Deflection Indicator
(Make from 2 x 2 x ⅛ cold drawn steel angle)
Fixated Reference Bar for Mounting Deflection Indicator
(Make from ¾ x 1" straight drawn aluminum bar, alloy 17 ST)
Milling cutter approx 3" dia.
Special Bolt With Beaded Shank
(Turn from ½ x 2" Hex Hd. Cap Sc.)

The Dial Indicator shown for deflection measurements is Ames Model 272 (B.C. Ames Co. Waltham, Mass.) provided with flat back and ¼" long contact point (P102). This model has a ¼" range and continuous reading double dials with revolution counter. Main dial is graduated for .001" increments and is numbered 0-100. Small dial with telltale indicates revolutions.

Dwg. 8—Assembly and parts of apparatus for measuring deflection of beam


62" Overall
Drill and tap Flange for 6-⅜x1" Flat Hd. Mach. Sc. ⅜-16-NC-2
Drill and tap Flange for 2-⅜x1½" Flat Hd. Mach. Sc. ⅜-20-NC-2
Half-Oval 3x⅜ hot rolled steel, 9'lg. Drill 13/16" dia. for 2 Mach. Sc.
⅛x8x18 cold finished steel plate. Drill 13/16" and C'sk. for flush fit with ⅜ Flat Hd. Mach. Sc.
Drill 6-13/16" holes for ⅝x1½ Hex. Hd. Cap Sc.
300#
HF Section 10 x 45.0 #x5'-2"
8.022
⅝ Bevel washer
Round corners to about ½" R
ROLLER BEARING UNIT (J) USED AT ONE LOAD POINT
(Make from cold finished steel rounds and flats)
ROLLER BEARING UNIT (K) USED AT ONE SUPPORT
STEEL BEAM FOR DISTRIBUTING LOAD FROM HEAD OF TESTING MACHINE TO DESIRED POINTS ON CONCRETE BEAM SPECIMENS
Drill and tap 2 holes for ⅜x1½" Hex. Hd. Cap Sc. ⅜-16-NC-2
Drill 2-1½ holes and C'sk. for flush fit with ⅜ Flat Hd. Mach. Sc. to permit bolting unit to bed of Testing Machine
See enlarged detail of spring tension
Lubricate with Vaseline
Section A-A
SPHERICAL BEARING UNIT USED AT ONE SUPPORT AND ONE LOAD POINT
(Make parts from cold finished steel. Grind Spherical Bearing Surfaces to perfect fit.)
Eye-bolt- Eye approx ½ O.D. x ⅜ I.D. Shank ⅜ dia. x 2½ long to center of eye with ⅜-20-NC-2 thd. ⅜ long and a finished Hex. nut
Drill and tap ⅜ deep for ⅜x1" Round Hd. Mach. Sc. ⅜-16-NC-2
Section B-B
Detail of Spring Tension Between Parts of Spherical Bearing Unit
0.675 Thick
1" Free Length
Make spring from No. 14 (OBO) steel wire. Both ends to be closed and ground flat.
Spring and Washer
(Scale: double size)

Dwg. 9—Details of apparatus for supporting and applying load to beam specimens


Lathe Spindle
File round corners
Turn smooth flat surface at each end and polish with fine Emery cloth
T (see table)
End mill flat surface wide for drilling conical hole dia. x deep for set screw (see notes 8,9 and 10).
Fixed End Block-JA
Identical Intermediate Blocks-JB and JC (Slightly adjustable on channel base)
Section A-A
Section B-B
Movable End Block-JD
Guide
Channel Base for Jig-JE
Fixed End Block-JA
Int Block-JB
Int Block-JC
Movable End Block-JD
Bushing for Block JA
Bushing for Blocks JB and JC
(All bushings are to be hardened)

DETAILS FOR MACHINING TYPICAL $\frac{7}{8}$ IN. DEFORMED REINFORCING BAR SPECIMEN READY FOR INSERTION IN MOLD FOR BEAM SPECIMEN

DETAILS OF PARTS AND ASSEMBLY OF JIG FOR DRILLING AND MILLING OPERATIONS ON BAR SPECIMENS

Dwg. 10—Details for machining $\frac{7}{8}$-in. deformed bar specimens and jig for use in drilling and milling operations on bar

TABLE AND NOTES ACCOMPANYING DRAWING 10

VALUES OF DIMENSION *T* FOR BAR AND *U*, *V* FOR BAR JIG

Specimen	Casting position	Length of embedment, in.	*T*	*U*	*V*
A	Erect	8	8.50	9.62	46.25
AA	Inverted	8	8.50	9.62	46.25
B	Erect	12	12.50	13.62	38.25
BB	Inverted	12	12.50	13.62	38.25
C	Erect	16	16.50	17.62	30.25
CC	Inverted	16	16.50	17.62	30.25

Reference marks on jig blocks

Arrows designated *X* and *Y* point to ends of blocks in jig which are to be marked by V-grooves, shown as dash lines, to indicate center line of holes for reference in mounting bar in jig as described in Notes 5 and 6 below. V-grooves should be about 0.02 in. wide and milled with same cutter as for V-groove detailed in DRAWING 5.

Notes on machining of bar

Proper machining of the deformed bar specimens under investigation constitutes one of the most important steps in carrying out the test program. The following notes indicate the sequence of machining operations and the use of the jig for drilling and milling the bar. These operations should not be conducted as a "production" job but as a "precision" job.

1. Center drill for lathe spindle each end of deformed bar specimen as nearly as possible on its longitudinal axis. If necessary, grind off lugs and seams for short distance at each end to leave round core which can be centered in chuck for center drilling.

2. Turn ends of bar to nominal ¾-in. diameter on lathe centers. It is very important to obtain *square shoulders* at *exact spacing* for proper location in jig for drilling and milling operations.

3. Turn ends of bar flat and to length indicated. Round corners with file and polish ends with fine emery cloth.

4. Install bar in jig by passing one end through blocks *JC* and *JB* into the hole of fixed end block *JA* and slipping movable end block *JD* over opposite end of bar. Block *JD* is clamped firmly in position by thumb screw when the turned shoulder at each end of bar is in intimate contact with the inner face of the adjacent end block.

5. Rotate bar in its end bearings until the parting line of the upper and lower halves of the bar (as rolled) coincides as nearly as possible with the horizontal center line *XX* of hole in each end block. (Center line is marked by V-grooves milled into inner face of each block). Cup-point set screws at the top of end blocks are then tightened to prevent rotation of bar.

6. Half-dog point set screws at the top and bottom of intermediate blocks *JB* and *JC* are now adjusted so that the parting line of bar coincides with center-line marks *YY* at ends of these blocks. This adjustment is very important. It provides for alignment of bars which are not perfectly straight and controls the accuracy of milling and drilling of holes which receive the points of set screws on yokes for elongation measurements. Cup-point set screws on opposite sides of each intermediate block hold the bar firmly within the center of the hole against lateral movements during milling and drilling.

7. When the bar is accurately positioned and firmly held at various points of the jig the ¼-in. diameter hole ⅜ in. deep at the extreme left end of bar is drilled. This hole is very important as it engages the special set screw in end plate of mold, as detailed in DRAWING 4, to control the position of the bar in the mold.

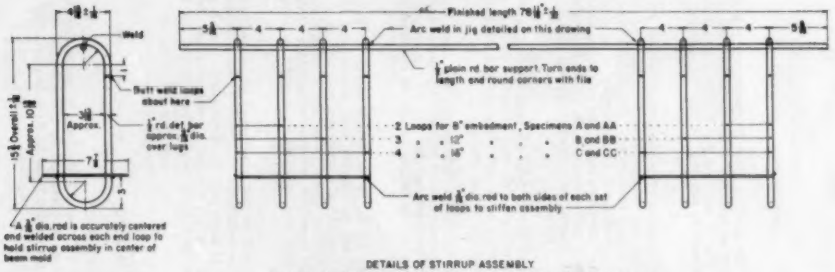
8. Through each bushing in blocks *JB* and *JC* mill a flat surface just deep enough to provide a flat area 5/32 in. wide for drilling gage points on parting line of bar. A Putnam No. A-78 high speed end mill ⅜ in. in diameter by 3¼ in. long with four flutes is contemplated for this operation. Keep curing edges sharp to reduce chatter and vibration to a minimum.

9. Through the bushings drill conical holes ⅛ in. in diameter and 1/16 in. deep into flat milled surfaces of bar. These holes are to furnish firm bearing for 90-deg cone-pointed set screws in yokes of gages for measuring elongation and slip at loaded end of bar as shown in DRAWING 7. In drilling these holes use specially ground ⅝-in. high-speed Morse Twist Drill Co. drills provided with cutting edges having a lip angle of 45 deg (instead of the usual 59 deg) and meeting in a point as sharp as possible consistent with good cutting action.

10. All milling and drilling operations should be so conducted as to reduce the cross-sectional area of bar as little as possible consistent with achieving precision in locating firm bearings for set screws of yokes at gage points. Obviously, once a given bar specimen has been positioned in the jig it must not be moved until all drilling operations are completed. Only straight bar specimens should be selected for these tests.

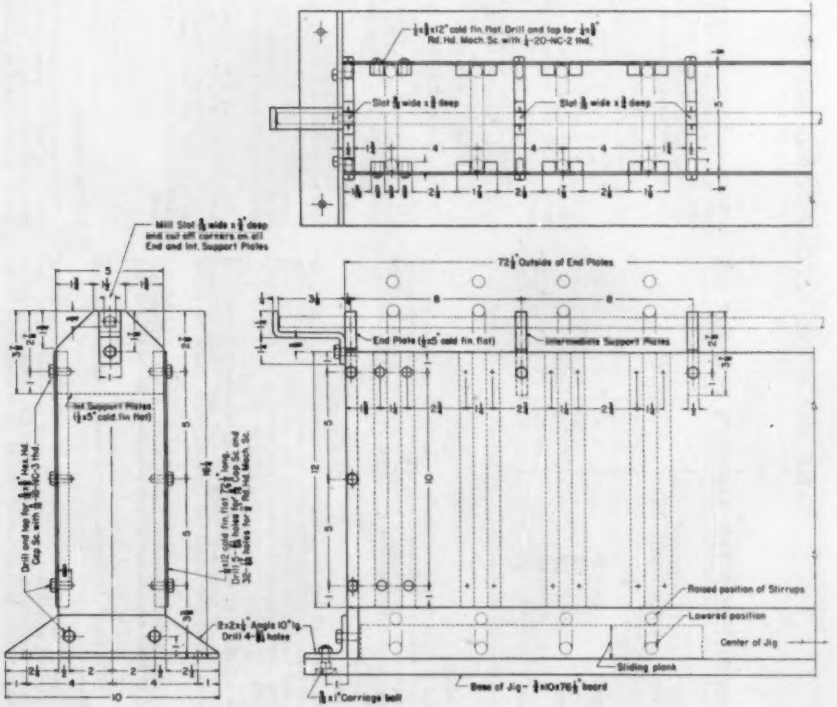
Rubber sleeves

These are approximately 1 in. O.D., ¾ in. I.D., and 3 in. long to cover the turned portion of bar between shoulder and end plate as detailed in DRAWING 4. The internal diameter should be small enough to provide a snug fit with bar and the ends should be cut square to a length of 3 in. ± 1/32.


Weld
Butt weld loops about here
Approx.
rd. def. bar approx. dia. over lugs
Overall
Approx.
A dia. rod is accurately centered and welded across each end loop to hold stirrup assembly in center of beam mold
Finished length
Arc weld in jig detailed on this drawing
plain rd. bar support. Turn ends to length and round corners with file
2 Loops for embedment, Specimens A and AA
B and BB
C and CC
Arc weld dia. rod to both sides of each set of loops to stiffen assembly

DETAILS OF STIRRUP ASSEMBLY

For a given length of embedment the stirrup assembly is identical whether the beam specimens are cast erect or inverted. The number of loops at each end varies with the embedded length of the deformed bar under test as indicated above.


Slot wide x deep
Slot wide x deep
Mill Slot wide x deep and cut off corners on all End and Int. Support Plates
Outside of End Plates
End Plate (cold fin. flat)
Intermediate Support Plates
Int. Support Plates (cold fin flat)
Raised position of Stirrups
Lowered position
Center of Jig
Sliding plank
Base of Jig-
Carriage bolt
Angle
Drill holes

JIG FOR HOLDING LOOPS DURING WELDING TO HORIZONTAL SUPPORTING BAR OF STIRRUP ASSEMBLY

Use a plank about 1 x5x65" resting on base of jig to support each set of loops until bar support is in place between end brackets of jig. By sliding each plank toward center of jig the loops will drop into normal position in contact with bar ready for arc welding.

Dwg. 11—Details of stirrup assembly and jig for holding loops during welding to horizontal supporting bar

Design of Concrete Floors on Ground for Warehouse Loadings*

By PAUL F. RICE†

SYNOPSIS

An application of the theory of beams on elastic foundations as developed by Timoshenko and Hetenyi. Representing actual soil conditions more closely, these solutions do not require the usual theoretical assumption of a tensile soil reaction for upward deflections. All results are tabulated to enable designer to select a proper slab thickness for a given load, soil, concrete strength, and layout. Criteria used for a proper slab thickness are avoidance of objectionable cracks and elimination of uplift causing visible vertical movements under warehouse-type vehicles.

Although the selection of thickness is based on nonreinforced concrete or concrete containing only distributed (temperature) reinforcement for shrinkage, moment values furnished may be used for design as reinforced concrete.

PROBLEM

A failure of concrete warehouse floors which has been repeatedly brought to the writer's attention by owners apparently occurs so often and in such regular, predictable patterns that it may be termed typical (Fig. 1 and 2).

Visible vertical movements up to ¼ in. often occur at the intersections of cracks or at normal joints crossing uplifted areas. These movements occur under relatively light moving loads such as unloaded vehicles and are so noticeable as to alarm owners. Triangular corner cracks often develop apparently as a result of loss of support and fatigue from flexure under the moving loads.

OBJECTIVE

The usual design procedure insofar as slab thickness is concerned is an arbitrary selection based on the designer's past experience rather than a rational consideration of expected loads. Where wheel loads are the controlling factor, numerous charts are available presenting the results of theory in a form suitable for practical use. At the same time equally applicable solutions for uniform load have been developed.[1] Application of these solutions has never become so widespread as those for wheel loads. One standard

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†Member, American Concrete Institute, Technical Director, American Concrete Institute, Detroit, Mich.

objection has been that these solutions are laborious, and the assumption of an elastic soil reaction capable of offering tensile (downward) as well as compressive (upward) forces obviously departs from the known physical nature of soil, hardly justifying the laborious solution. A considerable simplification in the application of these solutions results from the fact that, in the practical range, the ratio of slab thickness to length is small. Neglecting the discontinuity of load at the end removed from the section considered and the effect of joints or cracks at the column center line parallel to the section considered results in errors of about 1 percent.*

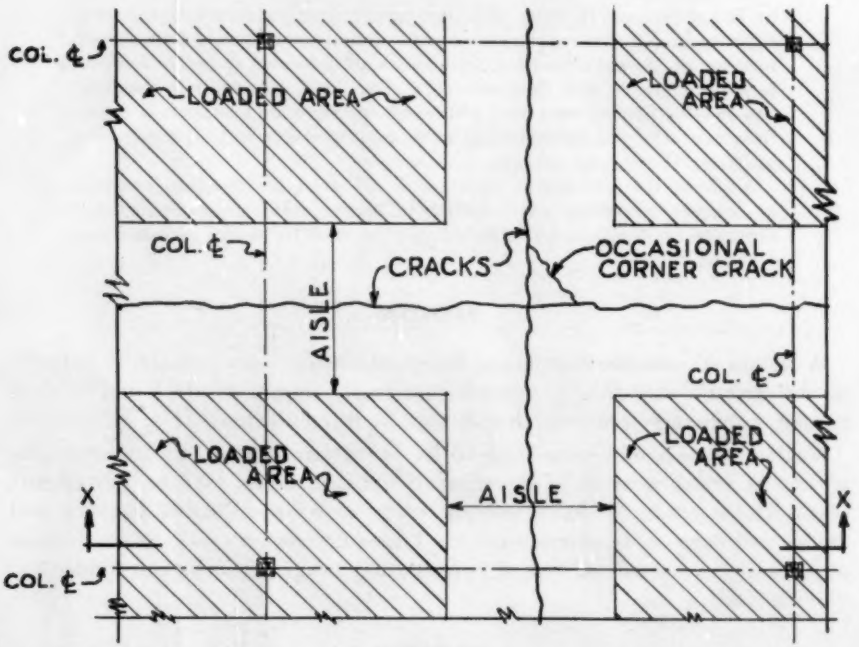


Fig. 1—Typical plan of warehouse loading arrangement showing typical crack pattern

Personal observation of failing as well as satisfactory floors has convinced the writer that, as long as the slab does not lift off its support, the objectionable conditions will at least escape notice in a casual inspection by owners. Such designs cause no complaint and are therefore satisfactory. The standard solutions in this desirable range then are those in which the tensile soil reaction is less than the dead weight of the slab. In this range obviously the assumption of an elastic soil will not conflict with the known physical properties of the soil.

*The writer is indebted to Alfred Zweig, Albert Kahn Associated Architects and Engineers, Inc., Detroit, who applied complete solutions to check the results presented herein.

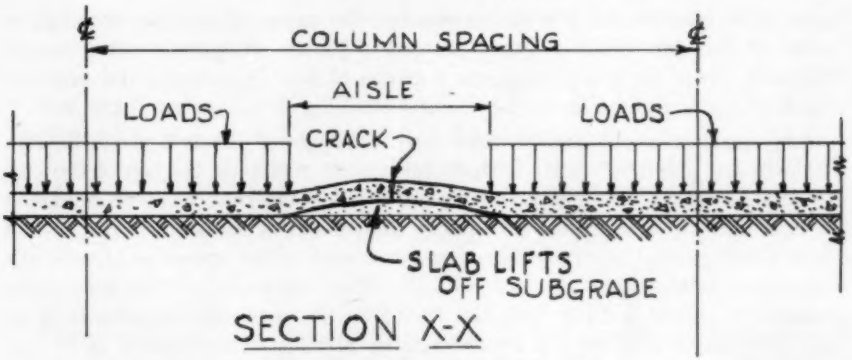


Fig. 2—Section through aisle showing typical crack

SOLUTION

The following tabulations are intended to present the maximum design moments as well as critical layout dimensions to cover the entire usual range of load, soil, and layout requirements. Slab thickness selections based on these limits will produce designs adequate for moments encountered, with no uplift.

It is seldom practicable to design or construct slabs on ground as reinforced concrete with top and bottom reinforcement in both directions to conform to such a design. Distributed (temperature) reinforcement, 2 in. to 1/3 the depth of the slab below the top, is effective to insure aggregate interlock at cracks and dummy joints, and to reduce random cracking. The following tables may be used to select slab thickness required for either non-reinforced slabs or slabs containing distributed (temperature) reinforcement. The moment values presented may be used to design reinforced slabs.

General description of tables

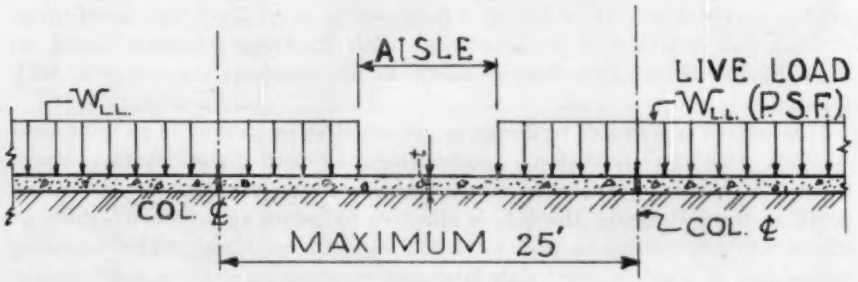
All tables are based on $f_c' = 3000$ psi. The reasoning in the use of a strength below the minimum 4000 psi generally recommended for resistance to abrasion is that most observed failures used a substandard mix, in some cases, barely 2500 psi. Tabulated values can be applied for any design strength by simply determining the equivalent thickness to maintain $E_c t^3$ a constant factor. Also if a user takes exception to the value $E_c = 1000 f_c'$, it is simple to replace the value with any other thought to be more suitable. For example, a 5-in. slab with $E_c = 3 \times 10^6$ gives $E_c t^3 = 375 \times 10^6$ which is equivalent to a slab $4\frac{1}{2}$ in. thick with $E_c = 4 \times 10^6$.

The commonly used layout of joints follows column center lines. For spacings not over 25 ft this arrangement is eminently satisfactory in controlling the shrinkage cracks. For spacings in excess of 40 ft common practice is to use intermediate joints or to expect shrinkage cracks. For spacings 25 to 40 ft, practice varies, and so does the occurrence of shrinkage cracks. Tables have been prepared therefore to include both systems in common use. A

third table offers a joint arrangement for the range of column spacings in excess of that for which a simple pattern of joints on column center lines is adequate. This third table suggests a means of sharply reducing the negative moments (causing tension on top) and controlling shrinkage cracks as well.

Only negative moments are considered throughout for two good reasons. The primary practical reason is that only cracks visible on the top of the slab are objectionable. A theoretical reason is the fact that positive moments under a uniform load are much smaller than negative moments. Only when wheel loadings are critical is the reverse true, and in the operation of ordinary warehouses wheel loads are not critical. This observation from theoretical grounds bears out a design practice based on the successful experience of at least one major engineering firm[2]—that is, placing reinforcement, if in one layer, near the top of the slab. Recommended practice in paving also requires reinforcement near the top surface.[3]

TABLE 1—MAXIMUM MOMENTS AND CRACKING LOADS*



Thickness t, in.	Soil k = 50			Soil k = 100			Soil k = 300		
	Ordinary good practice compaction in building construction			Fill material and compaction equivalent to ordinary highway practice			Selected material and compaction under laboratory control		
	Critical aisle width	Maximum moment, lb-ft per ft	Cracking load, lb per sq ft	Critical aisle width	Maximum moment, lb-ft per ft	Cracking load, lb per sq ft	Critical aisle width	Maximum moment, lb-ft per ft	Cracking Load, lb per sq ft
4	4 ft 4 in.	1.26w	475	3 ft 8 in.	0.90w	665	2 ft 8 in.	0.52w	1150
5	5 ft 2 in.	1.76w	535	4 ft 4 in.	1.25w	750	3 ft 4 in.	0.72w	1300
6	6 ft 0 in.	2.33w	580	5 ft 0 in.	1.65w	820	3 ft 10 in.	0.95w	1425
7	6 ft 10 in.	3.07w	600	5 ft 10 in.	2.15w	850	4 ft 5 in.	1.26w	1460
8	7 ft 4 in.	3.51w	685	6 ft 2 in.	2.50w	960	4 ft 9 in.	1.46w	1640
9	8 ft 0 in.	4.27w	710	6 ft 10 in.	3.00w	1000	5 ft 2 in.	1.74w	1750
10	8 ft 8 in.	5.00w	750	7 ft 4 in.	3.55w	1060	5 ft 7 in.	2.05w	1825
11	9 ft 4 in.	5.76w	790	7 ft 10 in.	4.05w	1120	6 ft 0 in.	2.36w	1925
12	10 ft 0 in.	6.56w	825	8 ft 4 in.	4.65w	1160	6 ft 5 in.	2.69w	2000

*NOTES: Contraction joints are on column center lines. Column spacing is a maximum of 25 ft to insure unbroken slab between joints.

Table is based on $f_c' = 3000$ psi; $E_c = 1000 f_c'$. Tensile stress due to shrinkage assumed equal to ½ MR = $0.15 f_c'/2$. Maximum w will cause tension of 225 psi. For a given t, maximum M is based on $Et^2 = 1000 f_c' t^2$. For E_c other than 3×10^6 psi, change t to maintain same Et^2.

Maximum M reduces for any change in critical aisle width. For aisles twice as wide, $M = 2/3\ M_{max}$. For alternately loaded bays, $M = ½\ M_{max}$ and will occur at distance from loads = ½ critical aisle width.

Apply desired safety factor to maximum w shown; reduce shrinkage proportionately by appropriate curing provisions in construction.

Table 1—All equations are for quantities at the center line of the aisle, denoted by subscript *c*.

$$\text{Moment } M_c = (-\,w/2\,\lambda^2)e^{-\lambda a}\sin\lambda a$$

Deflection $y_c = (w/k)e^{-\lambda a}\cos\lambda a$

Shear $Q_c = 0$

Angle change $\Theta_c = 0$

a = ½ the aisle width

k = soil modulus, lb per cu in.

w = unit load, lb per sq ft, with subscript *LL* or *DL* indicating live or dead load

$$\lambda = \sqrt[4]{\frac{k}{4\,E\,I}}$$

Due to the damped wave characteristic of the exponential-trigonometric expression, terms dependent upon the loaded length damp out as long as the loaded portion is approximately at least double distance a. ky_c cannot exceed w_{DL} if tensile soil reaction and consequent uplift off the ground are to be avoided. This condition will never occur as long as the slab weight is more than 1/15 the live load. A critical aisle width has been developed at which the moment reaches the maximum shown. Any deviation in aisle width will produce lesser moment. Also in a completely unloaded bay, a maximum moment will occur, which is approximately half that shown, at a distance approximately one-half the critical aisle width from the loaded area.

These equations are based on the existence of an unbroken slab across the aisle capable of moment resistance. Even construction joints must therefore be located only on the column center lines. A certain tension will exist in the slab due to shrinkage and some account must be taken of this tension. In the writer's experience at least one costly failure occurred before any load was applied, due entirely to this shrinkage and shrinkage warping of a slab jointed at 40-ft intervals. Since the majority of slabs with no special curing provisions do not crack between joints at 20 ft and do crack between joints at 40 ft, it seems reasonable to estimate the shrinkage tension as approximately one-half the concrete tensile strength for joint spacings under 25 ft. The "maximum" load tabulated is therefore that load which will cause a bending tension equal to the remaining concrete strength, based on a modulus of rupture equal to $0.15\,f_c'$. If exception is taken to this reasoning, the user can apply any desired factor directly to the given maximum loads assumed to cause moment cracking failure.

It is suggested that, in using Table 1 for uncertain aisle widths, the maximum moments shown be used. These moments would occur only for the critical aisle widths shown. For fixed layouts of floor area in which other widths of aisles are certain to be used, the moments can be reduced as shown, if desired.

It is further suggested that safety factors be applied to both sources of stress. The shrinkage stress may be reduced more or less proportionately to the reduction in load by appropriate curing requirements and limits on slump during placing.

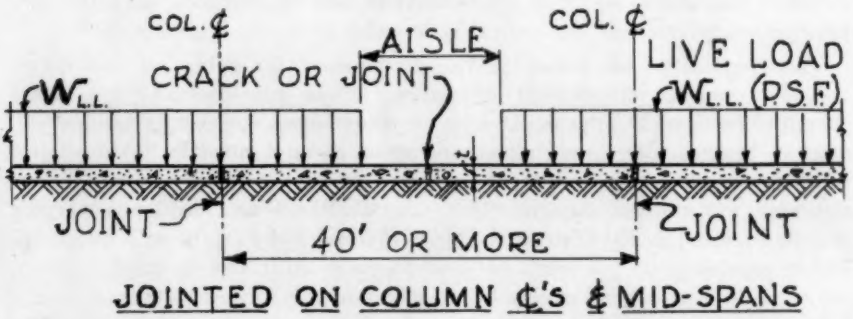
Table 2—The equations used here are as follows:

$$M_c = 0$$
$$Q_c = 0$$
$$y_c = (w/k)\, e^{-\lambda a} (\cos \lambda a - \sin \lambda a)$$

This table merely gives limits on aisle width as follows: (1) for no uplift at the center joint or crack under any live load; (2) for no uplift under live loads up to five times the slab weight. For the layout shown, i.e., joint or crack in the aisle, negative moment is not a controlling factor. It is herein implied that a satisfactory design is one in which the slab does not lift off its support. Vertical movement will of necessity occur under a moving live load, but it should not be visible, and so not objectionable. For unusually heavy wheel loads, the wheel load may conceivably control design in this case.

Table 3—Table 3 has been prepared as a result of examining the first two tables with a view to reducing both critical negative moment and shrinkage cracking. It is a suggested pattern of jointing only and does not pretend to

TABLE 2—SPACINGS PERMISSIBLE FOR NO UPLIFT*



Thickness t, in.	Soil k = 50		Soil k = 100		Soil k = 300	
	No limit on w	$w_{LL} \leqq 5w_{DL}$	No limit on w	$w_{LL} \leqq 5w_{DL}$	No limit on w	$w_{LL} \leqq 5w_{DL}$
	Maximum aisle width	Critical aisle width	Maximum aisle width	Critical aisle width	Maximum aisle width	Critical aisle width
4	4 ft 4 in.	8 ft 8 in.	3 ft 8 in.	7 ft 4 in.	2 ft 8 in.	5 ft 4 in.
5	5 ft 2 in.	10 ft 4 in.	4 ft 4 in.	8 ft 8 in.	3 ft 4 in.	6 ft 8 in.
6	6 ft 0 in.	12 ft 0 in.	5 ft 0 in.	10 ft 0 in.	3 ft 10 in.	7 ft 8 in.
7	6 ft 10 in.	13 ft 8 in.	5 ft 10 in.	11 ft 8 in.	4 ft 5 in.	8 ft 10 in.
8	7 ft 4 in.	14 ft 8 in.	6 ft 2 in.	12 ft 4 in.	4 ft 9 in.	9 ft 6 in.
9	8 ft 0 in.	16 ft 0 in.	6 ft 10 in.	13 ft 8 in.	5 ft 2 in.	10 ft 4 in.
10	8 ft 8 in.	17 ft 4 in.	7 ft 4 in.	14 ft 8 in.	5 ft 7 in.	11 ft 2 in.
11	9 ft 4 in.	18 ft 8 in.	7 ft 10 in.	15 ft 8 in.	6 ft 0 in.	12 ft 0 in.
12	10 ft 0 in.	20 ft 0 in.	8 ft 4 in.	16 ft 8 in.	6 ft 5 in.	13 ft 0 in.

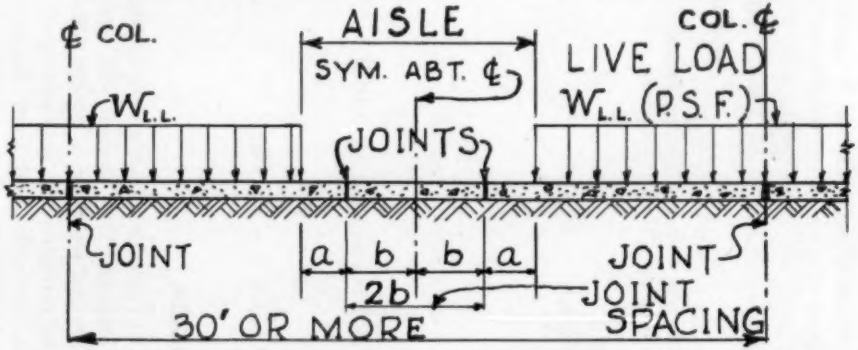
*NOTES: Columns spaced so far as to develop crack in midbay unless jointed to provide relief for shrinkage stress. Table is based on $f_c' = 3000$ psi; $E_c = 3 \times 10^6$ psi. For others maintain $E_c t^3$ constant by varying equivalent t.

w_{DL} = weight of floor slab; w_{LL} — uniform live load.

As long as live load is less than five times dead load, no uplift occurs. Critical aisle width is that width producing maximum uplift, at which $w_{LL} = 5w_{DL}$ will cause uplift equal to w_{DL}.

Maximum aisle width is that width below which no uplift occurs for any load. For larger aisles: at $1\frac{1}{3} \times$ critical width, uplift = 2/3 maximum; at double critical width, uplift = 1/5 maximum.

TABLE 3—SUGGESTED JOINT PATTERN FOR LONG BAYS*



JOINTED ON COLUMN ℄'S & EDGES OF AISLES

Thickness t, in.	Soil k = 50						Soil k = 100						Soil k = 300					
	a = 0 in.		a = 6 in.		a = 1 ft 6 in.		a = 0 in.		a = 6 in.		a = 1 ft 6 in.		a = 0 in.		a = 6 in.		a = 1 ft 6 in.	
	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment	Shear	Moment
4	1.40w	−1.27w	0.94w	−0.85w	0.28w	−0.26w	1.17w	−0.90w	0.72w	−0.55w	0.12w	−0.10w	0.89w	−0.52w	0.46w	−0.27w	−0.03w	+0.02w
5	1.65w	−1.77w	1.19w	−1.28w	0.48w	−0.52w	1.39w	−1.25w	0.93w	−0.84w	0.28w	−0.25w	1.06w	−0.72w	0.62w	−0.42w	0.06w	−0.04w
6	1.89w	−2.33w	1.43w	−1.76w	0.68w	−0.84w	1.59w	−1.65w	1.13w	−1.17w	0.43w	−0.45w	1.21w	−0.94w	0.76w	−0.60w	0.15w	−0.12w
7	2.18w	−3.07w	1.71w	−2.41w	0.94w	−1.32w	1.83w	−2.17w	1.37w	−1.62w	0.63w	−0.74w	1.40w	−1.26w	0.94w	−0.85w	0.28w	−0.25w
8	2.35w	−3.52w	1.87w	−2.81w	1.08w	−1.67w	1.97w	−2.53w	1.50w	−1.93w	0.75w	−0.97w	1.51w	−1.46w	1.05w	−1.01w	0.36w	−0.35w
9	2.57w	−4.27w	2.09w	−3.48w	1.29w	−2.14w	2.15w	−3.02w	1.69w	−2.37w	0.91w	−1.27w	1.64w	−1.74w	1.18w	−1.25w	0.47w	−0.50w
10	2.77w	−5.00w	2.29w	−4.13w	1.47w	−2.65w	2.33w	−3.54w	1.86w	−2.83w	1.06w	−1.67w	1.78w	−2.05w	1.31w	−1.51w	0.58w	−0.67w
11	2.98w	−5.77w	2.50w	−4.83w	1.67w	−3.23w	2.51w	−4.08w	2.03w	−3.30w	1.23w	−1.99w	1.91w	−2.36w	1.44w	−1.79w	0.70w	−0.86w
12	3.19w	−6.58w	2.71w	−5.59w	1.87w	−3.85w	2.67w	−4.65w	2.19w	−3.82w	1.38w	−2.40w	2.03w	−2.69w	1.56w	−2.06w	0.80w	−1.06w

*NOTES:
Condition (1)—Joint will transmit shear, no moment, to aisle. Maximum moment at center of aisle (lb-ft per 1-ft strip; w in lb per sq ft) for critical width joint spacing. Critical width in this case is the same as in Table 1. Maximum moment changes as noted in Table 1. Shear at joints in aisle (lb per 1-ft strip).
Condition (2)—Construction joints in aisle. No key or dowels. No deflection of joint will occur if distance a equals ⅓ critical aisle width. Lesser values of a give downward deflection to loaded side of joint.

represent the usual practice. The writer realizes that this arrangement of joints is entirely predicated on a fixed layout for the use of building, which is not too often encountered. It would seem to preclude any desirable flexibility in arrangement of the loads, but it must be admitted that this pattern is suitable for the average layout. Physical obstacles such as columns and physical equipment such as doors, loading docks, and cranes will more or less insure a similar layout for a storage area even though owners and materials stored may change. Two conditions are considered: (1) joints are presumed adequate for given shears by use of a designed shear key, a designed set of shear dowels, or simply aggregate interlock at a crack induced by tooling or sawing a dummy groove joint; and (2) joints are clean breaks with no shear capacity. The first condition is preferable of course to prevent unequal deflection on opposite sides of the joint.

Equations for condition (2) are the same as for Table 2.

Equations for condition (1) follow:

At joints in aisle:

$$M = 0$$

$$Q = (w/2\lambda)e^{-\lambda a}(\cos \lambda a - \sin \lambda a)$$

$$y = (w/2k)e^{-\lambda a} \cos \lambda a$$

At center between joints:

$$y_c = \frac{(2w)}{(k)} e^{-\lambda a}(\cos \lambda a - \sin \lambda a)\, e^{-\lambda b} \cos \lambda b$$

Deflection is always downward as long as b is less than ½ critical width and a is less than ½b.

$$M_c = \frac{-0.1612w}{\lambda^2} e^{-\lambda a}(\cos \lambda a - \sin \lambda a)$$

CONCLUSION

It is earnestly hoped that these tables will provide a usable basis for selecting the thickness and jointing plan for warehouse floor design. Of course no satisfactory floor can be assured by simply selecting a proper thickness. The whole problem of avoiding failures economically depends upon interrelated factors: design—slab thickness, strength, and joint pattern; and construction—slump, curing, and backfill material and compaction.

It is significant to the writer that the failures observed were cases in which low strength concrete, barely minimum thicknesses, heavy loadings, careless backfill, and no curing all combined to produce failure.

The range of soil values used in these tables does not in the least imply that backfill can be economically neglected and compensated for by increased slab thickness and strengths. Rather this range was used so that a designer can anticipate probable backfill results expected for each job. A wide range

of backfill material and effectiveness in compaction exists between small jobs with no compaction equipment or supervision and large projects with all available equipment and constant laboratory control.

The use of normal safety factors and a conservative estimate of backfill quality should result in safe, economical design.

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2. Albert Kahn Associated Architects and Engineers, Detroit, Mich.

3. "Design Considerations for Concrete Pavement Reinforcement for Crack Control," reported by Subcommittee IV, ACI Committee 325, ACI JOURNAL, Oct. 1956, *Proc.* V. 53, pp. 337-362.

Discussion of this paper should reach ACI headquarters in triplicate by Nov. 1, 1957, for publication in the March 1958 JOURNAL.

Multispan Prestressed Folded Plate Roof for Langendorf Bakery*

By JOHN J. DRISKELL†

SYNOPSIS

Folded plate roof over the production area of the new Langendorf Bakery in Los Angeles covers an area 171 x 365 ft without interior supporting walls. Roof is all lightweight concrete, 4 in. thick throughout except for thickening at prestressing anchorages. Structural design followed principles of accepted folded plate theory, modified to account for plate deflections and the application of prestressing.

INTRODUCTION

The recently completed roof structure over the production area of the new Langendorf Bakery in Los Angeles represents what is believed to be the largest example of folded plate construction yet attempted. The entire plant area comprises a total of 127,500 sq ft. The production area is entirely of fireproof construction, and the remainder of the plant has concrete walls with steel and wood roof construction. Production area comprises 62,300 sq ft; offices, 12,800 sq ft. Route trucks occupy 43,200 sq ft, and 9,200 sq ft are devoted to other uses. The general contract price was $952,500, a unit cost of $7.47 per sq ft of total plant area. Mechanical and electrical installations under the general contract amounted to $227,700, leaving a unit cost of the structure without these items of $5.70 per sq ft, which of course, includes roofing, glazing, painting, rolling steel doors, floor finish—including maple floor throughout the production area.

Of particular interest is the production area, the roof of which is all lightweight concrete, 4 in. thick except for extra thickening where prestress anchorages occur. The structural design follows the principles of folded plate theory (also known as hipped plate), as modified by the engineer to account for plate deflections and the application of prestressing techniques.

General layout

The production area in plan measures 170 ft 9 in. by 364 ft 9 in., the long dimension running north and south. All perimeter concrete bearing walls

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are 9 in. thick, heavyweight, 2500 psi material, with a constant height of 26 ft 4 in. above finished concrete floor level. Total interior support consists of two rows of spiral-reinforced 16-in. concrete columns, spaced 60 ft apart symmetrical about the long axis. Pairs of columns are spaced 52 ft on centers in the north-south direction, giving seven equal transverse bays. Each bay of the roof is composed of three 4 in. thick slabs having equal horizontal projections, the center slab being horizontal and the two outside slabs pitched on a slope of 4½:12 (Fig. 1). The five interior bays have no projections below the slab soffits for the full 170 ft.

Running continuously above the roof slabs, level with the outside walls, and along the two column lines are "inverted diaphragms" (Fig. 1) which serve as stiffeners for the shells along the lines of their interior support. They also pick up the plate reactions and carry them to the columns as a continuous haunched beam, and lastly contain prestressing cables needed to resist the "unbalanced" horizontal thrust from the two outside bay shells. In the latter it was necessary to project the two diaphragms below the shell to the valley level, 17 ft 10 in. above the floor, to adequately resist the combined stresses from shell action in an economical manner.

Advantages of prestressing

Previous experience with single span folded plate roof structures indicated that certain economies were attained with prestressing of the shell. Among the advantages are: (1) reduced shell deflection, usually halved; (2) elimination of necessary welding of main plate conventional reinforcing with the attendant danger of a weld fracturing in service; (3) elimination of the requirement for temperature steel transverse to slab reinforcing; and (4) either elimination or reduction of stirrup requirements. On previous folded plate roof designs, alternate plans had been prepared using both conventional and prestressing type shell reinforcing which showed the latter to be more economical at the time. This may not always be the case, however. Also at the time of first prestressing a folded plate roof structure of lightweight concrete, little was known or published about prestressing lightweight concrete. All the published recommendations urged the use of high-strength heavy concrete.

Thin shell roof structures, however, require a different approach in design thinking. For example, dead load is a major portion of the total weight to be supported. Significant economies may be attained through reduction of this item, even in view of the much greater cost of the lightweight concrete. Furthermore, shell flexural stresses are generally low compared to those encountered in pure flexural members, such as beams and girders. Hence it was considered feasible to employ prestressing techniques to reinforce the shell, provided the advantages gained through elimination of other types of reinforcing made it economically desirable.

The alternatives should generally be studied for each individual structure, weighing current costs of each type of system with the other advantages and disadvantages. Plastic flow of concrete is generally considered higher with

lightweight concrete, which would seem to increase the amount of prestressing materials otherwise required to overcome the loss of stress due to plastic flow. As indicated above, however, this factor can be re-evaluated for the case of post-tensioned work where two things work in the designer's favor. Prestress loss due to normal concrete shrinkage is almost entirely eliminated in post-

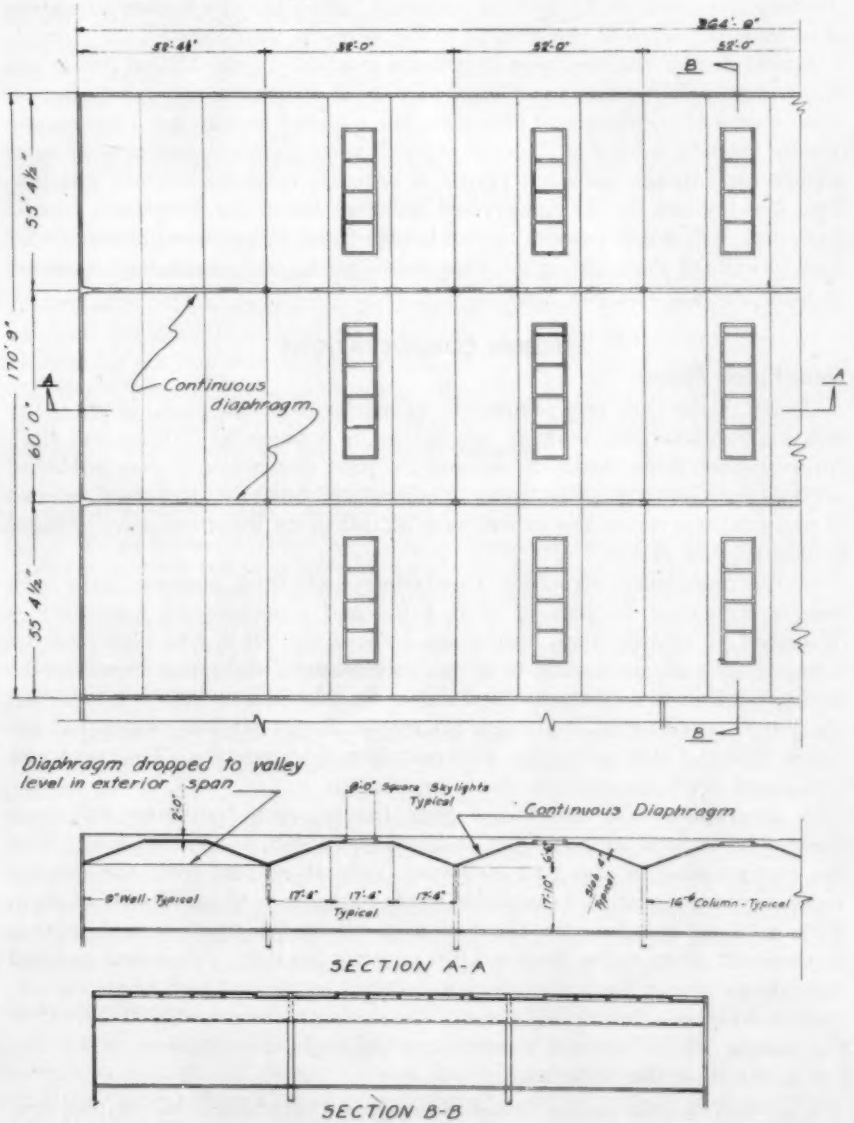

Fig. 1—Partial plan view and longitudinal and transverse sections

tensioning, and secondly, plastic flow is largely a function of the magnitude of the concrete stress as well as duration of stress. Therefore plastic flow is considerably less in the range of shell flexural stresses under prestress, which may range from 0.05 to 0.15 f_c' versus the more usual 0.45 f_c'. Losses due to plastic flow and creep of steel may be further reduced by temporarily overstressing the cables in the jacking operation, which has the further advantage of overcoming more of the friction of the wires in their sheaths.

Almost any of the prestressing systems available in the United States can be employed in prestressing folded plate roof structures, provided there is some means of enclosing and protecting the required anchorages. Anchorages usually must be located at the ends of the flexural members, and in some cases architectural design does not permit a concrete coverage beyond this line. This was the case in the Langendorf building; hence the Freyssinet system was employed, which permits the anchorage cones to be placed almost to the outside walls of the building, as there are no permanent projections necessary to hold the stress in the wires.

DESIGN CONSIDERATIONS

Folded plate theory

Design of the shell roof follows the principles of folded plate theory. The best known American work on the subject is a paper by Winter and Pei*. An extension of the theory to account for joint displacements was published in 1953 by Gaafar.† This latter modification, however, had been applied in practice long before the papers were published by those engineers engaged in this type of design.

In the Langendorf structure, twenty-one 4 in. thick concrete slabs each span a horizontal distance of 17 ft 4 in., and complete slab continuity is developed at all fold lines. Slabs are designed for 20 lb live load plus the actual dead load, amounting to 36 psf on horizontal slabs and 39 psf on the horizontal projection of the inclined slabs. Skip loading is employed to obtain the initial, unadjusted, maximum transverse design stresses, which fall between 1000 and 1100 psi for the 3000-psi lightweight concrete. The transverse continuous slabs are conventionally reinforced.

In accordance with the folded plate theory, each transverse slab span receives its vertical support from each adjoining slab, at the fold line. With the configuration of this roof structure, vertical support from the exterior walls or interior columns is available at the valley lines of each barrel or shell. With a 12 psf live load for the roof as a whole, the total vertical support requirement along valley lines is 2.600 kips per lineal ft. Thus each inclined plate (in an interior bay) must carry in its plane an inclined load whose vertical component equals the vertical shear of the sloping transverse slab at the valley fold line, i.e., $V \times$ cosecant a (where a is the angle of inclination) $= 3.7$ kips per ft, which is the plate load.

*Winter, George, and Pei, Minglung, "Hipped Plate Construction," ACI JOURNAL, Jan. 1947, *Proc.* V. 43, pp. 505-532.

†Gaafar, Ibrahim, "Hipped Plate Analysis, Considering Joint Displacements," *Transactions*, ASCE, V. 119, 1953, p. 743.

Actually, then, each barrel working with its neighbor forms a three-span continuous flexural member, with end spans of 55 ft and an interior span of 60 ft. The distribution of moments along the member is quite similar to that of ordinary continuous beams. The plate can be isolated and treated as a continuous three-span beam. Moments, shears, and reactions are determined. At this stage we have a maximum negative moment of 1270 ft-kips and maximum positive moment of 917 ft-kips, with reactions to columns (in plane of plate) of 240 kips at the interior, and 85 kips at the exterior support. These are not design stresses, but are required for the analysis of the shell action, or interaction and continuity of the adjoining plates forming the shell. This analysis substantially follows the procedure outlined by Winter and Pei*.

Application of prestressing

The folded plate analysis must be amplified to include the effect of the prestressing. The amount of prestressing required and the placement of cables is probably best determined by trial, although experience leads to an economical solution without excessive labor. In this type of application it was considered undesirable to place cables parabolically so as to follow the stress trajectory (although we have done so in certain pretensioned applications of non-shell structures), for the following reasons:

1. Change of slope of cables was so large that excessive frictional losses would occur, resulting in widely varying resultant axial forces.
2. Taking account of friction losses in design calculations leads to complicated mathematical notations and when the rather questionable distribution of such friction losses is considered the reliability of the final analysis is subject to great doubt.
3. To obtain the required prestress force in interior regions would have required additional prestressing material to make up for the friction losses. Therefore it was decided that straight cables in the inclined plate would be most economical, as friction losses would then be low.

The preliminary plate analysis indicated that a poststressing force of about 450 kips in each inclined plate would be required to eliminate all tensile stresses in the shell. (At the time of designing this structure, the Freyssinet 0.196-in. wire available had an ultimate strength of 220 ksi, whereas it is now 250 ksi.) Using 12-wire Freyssinet cables with an accepted working stress of about 125,000 psi it was found that ten such cables would be required to eliminate all tensile stresses in the shell (Fig. 2). Shell stresses were computed to be a maximum of 900 psi compression at interior supports. All other longitudinal direct stresses were less than this figure.

Shearing stresses

However, longitudinal shear between plates at the ridge fold line for interior shells was as high as 150 psi. These shearing stresses had to be combined with the vertical shear in the inclined plates to obtain the actual shear at any point in the structure. Shear stresses were combined following Winter and

*See footnote, p. 118.

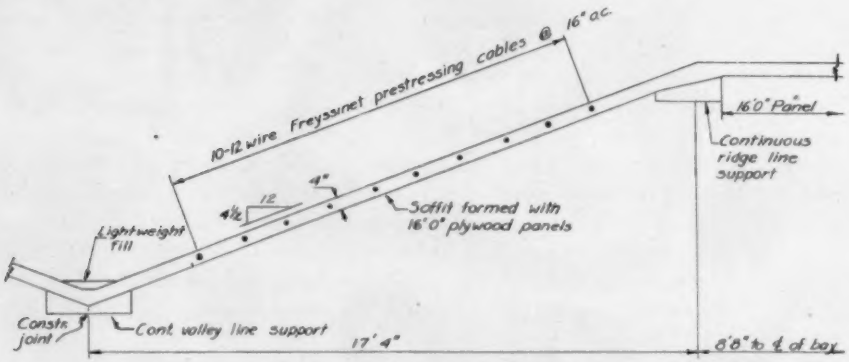

Fig. 2—Section of typical interior inclined slab, with portion of adjoining horizontal slab, showing location of prestressing steel only

Pei's methods. The direct compressive force in the plates was not sufficient to keep diagonal tension stress less than $0.03\ f_c'$; hence the regions of high shear near the supports required shear reinforcing. This was accomplished by increasing the transverse bottom slab steel by an amount sufficient to resist the slab bending stresses and the plate shearing stresses. The ends of such steel were provided with stirrup hooks over continuous tie bars in both the ridge and the valley.

Along the ridge line at the extreme outer ends of each shell there is an additional longitudinal shear over and above that developed in normal folded plate action, induced by prestressing only the inclined plates. If it had been feasible to locate the center of gravity of the cables one-third of the distance up the inclined plate from the valley, this would not be so. But for reasons mentioned before the ten cables were placed symmetrically in the plates, which induces longitudinal shear between them and plates not containing prestress cables. Since these shears were rather high they were resisted by a double layer of inclined #6 stirrups at each ridge, at the outside walls only.

Variation in exterior bay design

The exterior bay design followed more closely the procedures already available in the cited literature. The outside inclined plate receives vertical support from the wall, but no horizontal support. In this structure, a multitude of different wall conditions below the eave line led to devising a special detail whereby the slab edge was *simply* supported by the two side walls, in effect a hinged connection capable of transferring only seismic shears to the walls. Vertical and horizontal components of the slab edge reaction were determinable from the transverse continuity analysis by taking moments about the first ridge joint, in a manner similar to that employed in determining redundant reactions in rigid frames. While transverse slab positive bending moments are larger in the two exterior slabs, shell stresses in these bays are somewhat lower than for interior barrels.

Skylights

To provide as much natural daylight as possible to the interior, four 8- x 8-ft openings were made in the horizontal plate of each shell span in all five interior bays. With heat-resistant glass in all skylights, good general illumination was obtained.

The skylight framing presented no unusual problems, as they were confined to the horizontal plates of interior shells (Fig. 3). Here there are no unbalanced

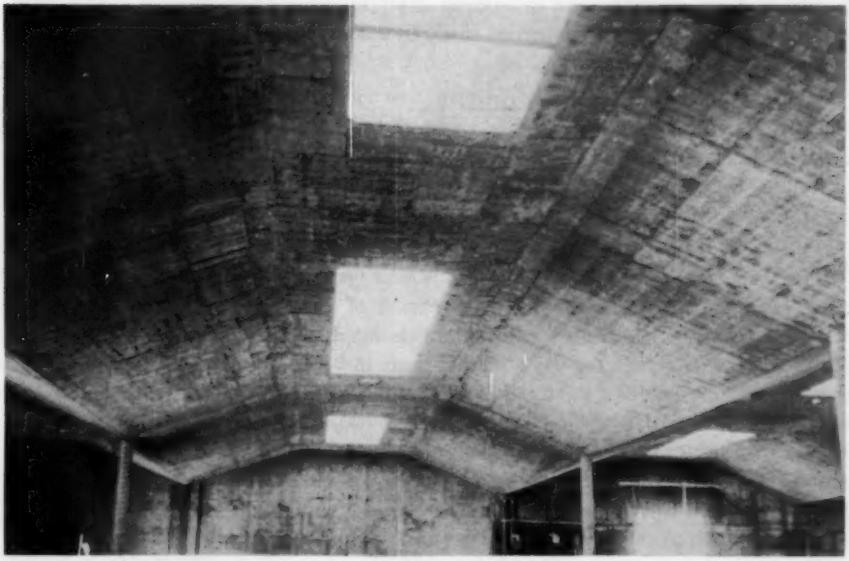

Fig. 3—View of one interior bay after prestressing and form stripping. Interior bays are each provided with three 8- x 32-ft skylight bands. Roof has no projecting ledges or framing to catch dirt

horizontal forces of any magnitude, so Vierendeel action is absent. Concrete struts between the individual openings were so proportioned that they had the same moment of inertia as the slab they replaced. This avoided discontinuities in the continuous slab action. Their size was about the minimum that would be desirable for good concrete placing and their height was just sufficient for the curb requirements of the skylight framing they received. They were reinforced to resist combined bending and direct compression.

Diaphragm design

At the outer shell ends, east and west walls are available to contain the tie necessary to resist horizontal thrust of the shells. Under symmetrical roof loads, each interior shell has only its horizontal thrust resisted by its neighbors. The exterior bays generate an unbalanced horizontal thrust. This thrust is greatest, naturally, at the two interior lines of support where the diaphragms are located (Fig. 1). The plate reactions at these points are 240,000 lb in the

plane of the inclined slab. All plate reactions are transferred to the diaphragms through the junction of the inclined slabs to the diaphragms.

Inclined reactions are transferred from plates to diaphragms by shear in the concrete, assisted by dowel action of the diaphragm stirrups, which extend nearly to the soffit of the slabs, thereby supporting them vertically also.

The five interior diaphragm spans are loaded principally with a partial vertical load. They are designed as continuous haunched beams with a partial uniform vertical load extending over the outer third of each span. The magnitude of each of these partial loads is the vertical component of the plate reaction.

The two diaphragms each contain five prestress cables running the full length of the building, provided to resist the unbalanced thrust of the outside bays, or shells. Cables were laid in a straight line the full length of the diaphragms, with their center of gravity 7 in. above the bottom of the 2-ft-deep portion of the diaphragms. A second analysis of the continuous beams was made to obtain the design stresses caused by prestressing.

The two exterior spans of the diaphragms present special problems, created chiefly by the unbalanced horizontal thrust of the shells they support. The resultant of this horizontal component is applied to the diaphragm at the midheight of the inclined slab which then induces a large positive moment throughout the exterior span. It would not have been possible to adequately resist this moment in the regular haunched beam shape of the interior spans. The diaphragm was accordingly dropped to the valley level in these exterior spans, analyzed for continuity with the interior spans, and additionally reinforced with seven prestress cables whose center of gravity was 12 in. above the bottom of the prismatic section. The chief function of these cables, it should be noted, is to counterbalance the eccentric thrust of the continuous cables near the top of the beam. The approximate axial stress in the diaphragm is then zero. Conventional reinforcing carries the vertical and variable live loads.

Among the factors influencing the decision to employ prestress cables running the 365-ft length of the diaphragms to resist unbalanced thrust was the fact that conventional reinforcing steel, to be made continuous, would need many butt welds in that length, with the possibility that one of them might inadvertently fail. Also, there would be over 3 in. elastic elongation in such tie steel, which was not considered desirable.

Negative moments over interior columns were about 1000 ft-kips. The diaphragm is 8 ft deep at the columns, 13 in. thick throughout, and is conventionally reinforced for such stresses. Maximum unit vertical shear was 85 psi; #4 U-stirrups were used throughout the haunches, however, and performed the additional useful function of positively supporting the shell slabs.

Buckling

Buckling of the thin shell was considered but found not critical. The system is roughly analogous to the prestressed column, in which lateral

deflections caused by external loads are counteracted by the internal prestress force. In this roof the center of gravity of the cables is slightly below the neutral axis of the entire section, so without gravity stresses the shell compressive stresses due to prestress are nearly uniform.

CONSTRUCTION

Concrete mix

Concrete for the thin shell roof was made with an expanded shale aggregate coated with a ceramic glaze. These lightweight particles have greatly reduced absorption, resulting in a mix that is far easier to control. The specifications called for a 3000-psi concrete, with a density of 95 lb per cu ft; cement factor, 6 sacks; W/C ratio, 6 gal. per sack. Mix proportions by weight were: cement, 1; lightweight sand, 1.71; 9/16-in. size, 0.63, 1-in. size, 0.78. Sand and coarse aggregate were manufactured by Rocklite Products Co., Ventura, Calif. Saturated cylinder weights ran just over 100 lb per cu ft. Plastiment was used as an admixture, at the rate of ½ lb per sack of cement. Seven-day strengths ran about 3000 psi, with 28-day breaks testing well over 4000 psi, and some cylinders testing as high as 5300 psi.

Low axial compressive stresses in the prestressed shells seemed to justify the adequacy of 3000-psi concrete. High shearing stresses at fold lines were taken care of by shear reinforcement. Heavyweight 5000-psi concrete was used in the thickened 12-in. strip encasing the anchorage cones at the two ends of the shell, in order to better withstand the high local stresses caused by the jacking.

Construction sequence

In construction, the walls, floor, and columns were placed first. Forming of the roof was clearly indicated on the plans and in instructions to the bidders. Construction joints at valley lines were indicated, with the intention of placing one complete 52- x 170-ft bay in one day's operation, about 120 cu yd of concrete. It was necessary to support the shell along the fold lines until the shell was prestressed, hence a detail was provided indicating this portion of form and shoring to remain in place after slab forms were dropped to be reused in another bay. This slab form width was deliberately set at 16 ft, so all reusable slab forms utilized uncut plywood panels of standard size. The contractor elected to provide sufficient form material to completely form two adjacent bays. This enabled him to set all steel in one bay complete (Fig. 4), then while concrete was being placed, and during the 7-day slab curing period, steel could be placed in the next bay. In this manner, slab forms could be used three to four times and there was a continuous flow of work. Ideally, this resulted in a bay being cast every 10 days. After use in forming the roof the plywood panels, still intact, were used for sheathing the wooden roof portions of the plant outside the production area, resulting in an exceptionally low roof forming cost.

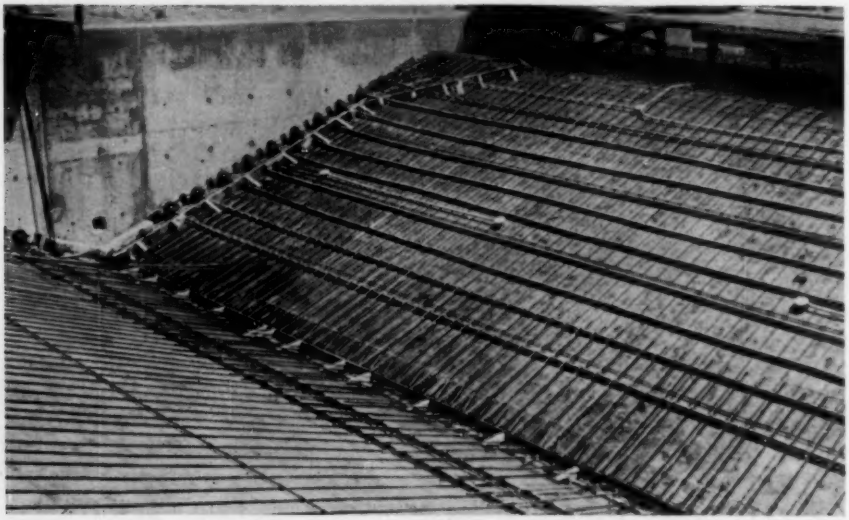

Fig. 4—Portion of roof forms with reinforcing in place. Freyssinet anchorage cones placed in holes through outside walls. Temporary wooden bulkheads in front of cones form dam for anchorage block of 5000-psi concrete. Note crossed shear reinforcement in ridge at end wall

No difficulty was experienced in placing the concrete due to the 4½:12 slope of two-thirds of the slabs. The slab steel and spacers provided a good key to prevent sliding of the concrete, even under vibration. For ease of finishing and to prevent segregation, the slump was kept from 1 to 3 in., which also helped prevent sliding. Concrete buggy runways were provided along each horizontal slab. Concrete was hoisted from the transit mixers on the ground by cranes which dumped it into hoppers at the roof level; the buggies discharged the concrete into movable chutes to points of deposit (Fig. 5). Concrete was then sparingly vibrated, screeded, and given a medium smooth finish with wood trowels. Curing was by means of sprayed-on liquid curing compound.

Prestressing practices

When the shell concrete attained its design strength, in many cases in 7 days, it was ready for prestressing. After prestressing of each shell, the fold line supports were removed for reuse. The bottom set of cables in the full depth diaphragm of the first exterior bay was also prestressed at the time of prestressing the shell cables. Progressively stressing each shell then, it was possible to remove the fold line supports, but it was still necessary to provide intermittent support along the diaphragms until the entire roof had been concreted. At this time, of course, the diaphragm and wall cables could be stressed, after which all shores were removed and the entire structure became self-supporting.

Anchorage cones were placed almost to the outside walls of the building where holes through the wall, following the trace of the shell roof (Fig. 4, 6), provided access for the jack. The holes were later dry packed and finished to match the outside surface.

All prestress cables were pressure grouted after stressing and anchoring. The writer is convinced that this practice is desirable. Reliance is not placed permanently on the anchorages, even though they are made adequate to permanently maintain the prestress force. Conceivably they might be damaged in the future, but if through grouting they are no longer necessary, no harm results. Furthermore, complete interaction of the prestressing steel and the concrete is assured by proper grouting of the wires in their sheath, resulting in an increased factor of safety against accidental overloading. Also, if through unanticipated excess friction losses the full design prestress force is not attained at some point, the steel is not stressed to its design value and thus has reserve strength to act in part as conventional concrete reinforcing without overstress.

Drainage

Roof drainage was accomplished by placing lightweight (35 lb per cu ft) fill in the valleys to form saddles. Drains and overflow drains were placed through the inverted diaphragms to pass the water to the outside walls where conventional collectors and downspouts carry it off.

COSTS AND ADVANTAGES

The cost of the completed roof, structure only, in place was about $1.50 per sq ft. This is comparable to the cost of fireproofed wood and steel com-

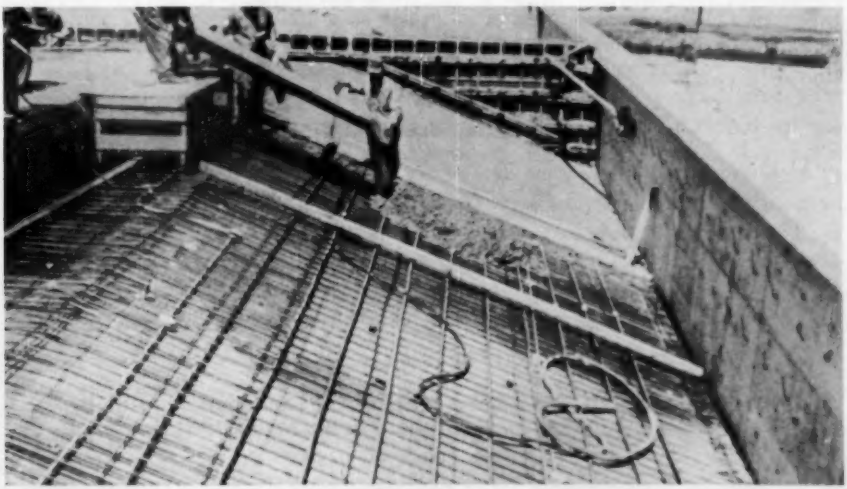

Fig. 5—Placing concrete on sloping plate. Forms for inverted diaphragm can be seen in background behind workmen

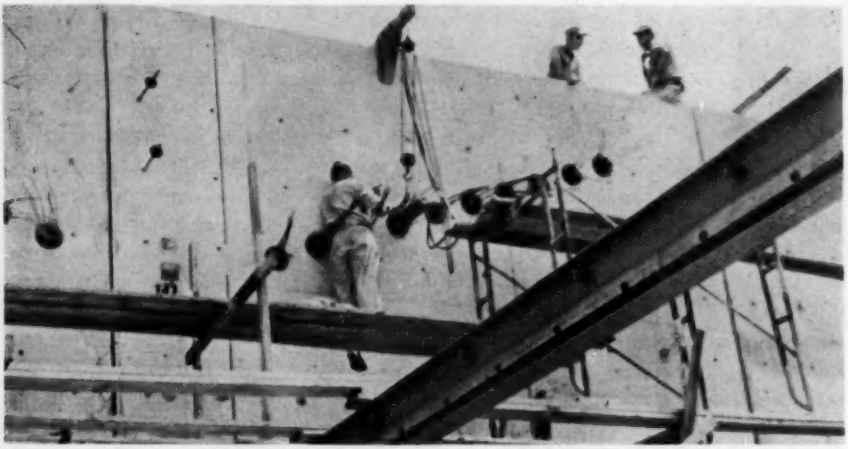

Fig. 6—Prestressing operation. Two jacks were used for each cable, stressing simultaneously from both ends. Man on scaffold measures elongation of cable wires

binations, except that the latter would require columns spaced north-south at about 20 ft instead of the 52 ft used. Otherwise supplementary girders or trusses would have been needed, which would have increased the cost.

Further advantage of the incombustible construction is that it allows unlimited area under one roof without interior firewall separations; lastly, the concrete roof is a rigid horizontal diaphragm in the West Coast earthquake sense, which means that no supplementary horizontal bracing is needed to resist earthquake forces, such as would be required with many other alternate types of construction. From a sanitary standpoint, the folded plate roof structure is unexcelled; there are no projecting ledges or framing of any sort that will catch and hold dust and dirt (Fig. 3).

ACKNOWLEDGMENT

Architects for the Langendorf Bakery were Bennett & Bennett, Pasadena. Structural engineer was John J. Driskell, San Gabriel; mechanical engineer, H. F. d'Autremont, Los Angeles; electrical engineer, G. P. Byrum, also of Los Angeles. General contractor was R. J. Daum, Los Angeles, with Norman Barber as his general superintendent. The author wishes to express his appreciation for the able assistance of his associate, Robert E. Parker, structural engineer.

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Title No. 54-9

Effect of Tobermorite on the Mechanical Strength of Autoclaved Portland Cement-Silica Mixtures*

By L. DEAN SANDERS and W. J. SMOTHERS†

SYNOPSIS

Mechanical strength of steam-cured neat portland cement and cement-silica mixtures was evaluated by utilizing metal test fixtures designed to simulate the ceramic and metal parts of a high-tension porcelain insulator. Five silica materials were tested as additives to cement: two noncrystalline materials, and three samples of crystalline quartz of differing particle sizes. Conditions of steam curing necessary to bring about optimum strength of the cement-silica mixtures were determined. X-ray diffraction analysis was used to identify the crystalline phases present after curing the cement mixtures. Amount of the platy phase of the hydrate tobermorite formed by the steam curing was found to be a definite indication of the mechanical strength of the cement-silica mixtures.

INTRODUCTION

Results of this investigation came about through a study of high-strength cements to be used in the assembly of high-voltage porcelain insulators of the suspension type. It was desired to obtain a cement product with strength superior to that of moist-cured portland cement.

Various references cite the fact that steam curing of mixtures containing portland cements and fine-grained quartz (or siliceous materials) has a beneficial effect on mechanical strength. Brown and Carlson[1] worked with mixtures of 60 percent white portland cement and 40 percent quartz sand (minus 200 mesh). These mixtures were autoclaved 20 hr at 170 C (100 psi, gage pressure) after a 24-hr setting period. Tensile strength of the resulting product was 980 psi compared to 420 psi for neat portland cement given the same treatment. Menzel[2] reported compressive strengths of 22,000 psi for 60-40 cement-quartz mixtures cured 3 days at 170 C, and 18,000 psi for 70-30 mixes cured 1 day under the same conditions. He obtained strengths of 13,500 psi for neat portland cement cured 28 days in a moist room. Steam-cured portland cement without the presence of quartz did not develop a strength equal to the moist-cured neat cement.

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DESIGN OF TEST FIXTURE

A cutaway view of a typical assembled suspension insulator is shown in Fig. 1. The porcelain part is indicated by *A*. For use on an electrical high-tension system it is necessary to combine the ceramic part with metal parts *B* (the cap) and *C* (the pin). Neat portland cement *D* is used to provide bond between the ceramic and metal. To facilitate bonding, coarse-grained ceramic material *E* (called "sand" in the trade) is fixed to the porcelain during its manufacture and thereby the unit presents a roughened surface to the cement.

To simplify testing of cement compositions, metal test units (Fig. 2) were designed to simulate the essential internal features of the suspension insulator head. Inside diameter of the test unit is 1¾ in. with eight threads per in. The same metal pin *C* shown in Fig. 1 is cemented into the metal test fixture shown in Fig. 2. With this metal test unit the force necessary to pull the pin out of the assembled unit is more representative of cement strength than would be the breaking strength of the assembled insulator when tested to destruction in a tension-type test.

RAW MATERIALS

Portland cement

Type I portland cement was used throughout this investigation. The following specifications were given by the manufacturer:

Chemical analysis:

SiO_2	22.5
Al_2O_3	5.1
Fe_2O_3	2.3
MgO	3.3
SO_3	2.0
Loss on ignition	0.8
	36.0

Specific surface: 1740 sq cm per g

The amount of CaO present in this cement was not specifically stated, but in calculating the CaO/SiO_2 ratios later in this report, it is assumed to be the remainder of the composition, or 64 percent.

Powdered silica

The siliceous materials are classified as shown:

1. A chemically-precipitated silica containing no crystalline material (as determined by x-ray diffraction analysis); average particle size 0.022 micron. This material will be designated as "silica gel."
2. Ground quartz with average particle size 12.7 microns, designated as 12.7-micron quartz
3. 18.6-micron quartz (ground quartz)
4. 20.0-micron quartz (ground quartz)
5. Diatomaceous earth

Table 1 indicates particle sizes of the three kinds of ground quartz.

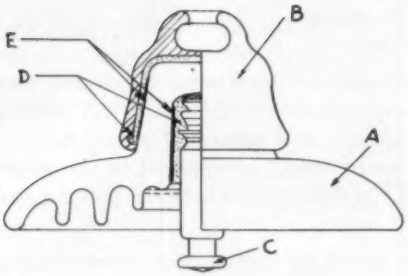

Fig. 1—Cutaway view of typical assembled suspension insulator:

A. Porcelain
B. Metal cap
C. Metal pin
D. Neat portland cement
E. "Sand grains"

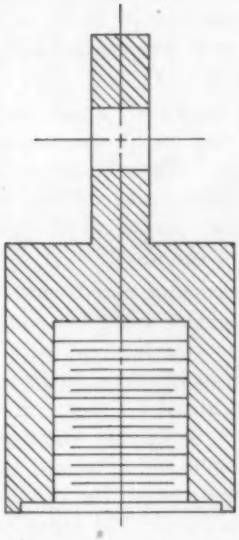

Fig. 2—Cross section of metal test unit

EXPERIMENTAL PROCEDURE

All cement batches were mixed in a Hobart C-100 mixer with enough water to bring them all to the same consistency. The cement was then poured into the pinholes of the test fixtures and the pins inserted. Cardboard rings were used to keep the pins centered in the holes during the curing cycle. The standard steam curing procedure began with a 24-hr setting period in air at room temperature followed by a 24-hr period in an autoclave at 182 C. This temperature corresponds to a saturated steam pressure gage reading of about 138 psi. No attempt was made to control heating and cooling rates during the steaming cycle. About 15 min was required to raise the temperature inside the chamber from ambient to 182 C. At the end of the cure the steam was released within a period of about 5 min. The chamber was then opened and the test units were allowed to cool to room temperature in the chamber.

Seven days from the date of pinning the pins were pulled from the test units and the strength values observed. The usual number of samples in a

TABLE 1—PARTICLE SIZE DISTRIBUTION OF THE THREE VARIETIES OF GROUND QUARTZ USED

Quartz (Average particle size)	Percent finer than given size (microns)					
	32	17.5	10	6.9	4	2.6
12.7 micron	85.2	62.0	40.8	31.0	18.6	11.5
18.6 micron	71.0	48.8	31.2	22.4	13.8	9.0
20.0 micron	70.2	45.0	29.1	20.5	12.2	7.7

group was four or five. In later tests certain phases of the standard steam curing cycle were varied and the effect on strength of a given cement mixture was noted.

For comparison, a test was run in which neat portland cement was used with a 4-day cure in production moisture chambers directly following the pinning operation. These moisture chambers operate in a saturated atmosphere at 46 C. This treatment was designated "moist cure," as opposed to the aforementioned "steam cure." The same 7-day period elapsed between pinning and testing of the units.

The raw and cured cements were examined by means of a Norelco x-ray diffractometer using Cu K-alpha radiation with a nickel filter. The unit was originally standardized with both silicon and quartz samples. Because one of the most important x-ray diffraction peaks reported herein falls at a 2-theta value of about 7.7 deg, corresponding to a d-spacing of 11.4 to 11.5A, it was necessary to further standardize the diffractometer at these low angles. A sample of calcium lactate USP powder was chosen for a standard material. A powder pattern for this material was run and compared with the pattern given on the ASTM index card 5-0101 for calcium lactate of USP purity or better. The two patterns were in agreement well within the expected limits. The reported peak at 11.8A fell at 11.85A and the reported peak of 9.85A fell at 9.93A. Assuming that the ASTM card for calcium lactate is reasonably accurate, it is felt that the reported spacing for the cement materials in this low angle region should be accurate within ± 0.1A.

The cement samples to be analyzed were crushed in a steel mortar to pass a 100-mesh screen. The sample was then further ground for 15 to 20 min in a Fisher mortar grinder. The powdered sample was prepared for x-ray diffraction according to the procedure developed by McCreery.[3]

RESULTS OF MECHANICAL STRENGTH TESTS

The investigation was begun by establishing a standard pulling strength value for the neat portland cement cured 4 days in the production moisture chambers at 46 C. This standard value was established at 28,300 lb.

After the standard procedure for high-temperature steam cure was established, five varieties of silica were tested in various proportions with the portland cement to determine the maximum strength attainable for each variety (Fig. 3). The strength of each cement mix containing a constant amount of the various silica samples is approximately proportional to the average particle size of the silica used. Addition of silica gel and diatomaceous earth considerably decreased the strength compared to that of neat portland cement given the same high-temperature cure.

The powdered 12.7-micron quartz composition exhibited maximum strength in the 35 percent by weight mix corresponding to a C/S* molar ratio of 0.9.

*C/S is used throughout as simplified form of CaO/SiO_2.

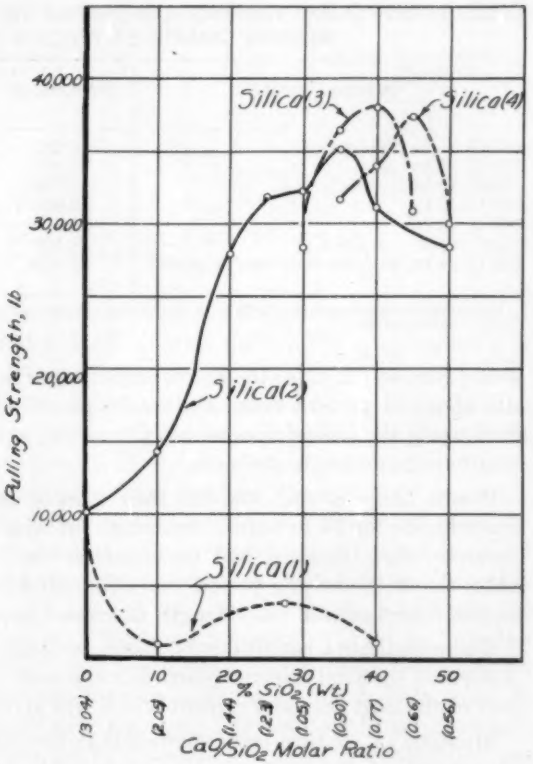

Fig. 3—Curves showing relationship of mechanical strength to percent addition of various siliceous materials. Pulling strength of moist-cured neat portland cement was 28,300 lb

Compositions containing the 18.6- and 20.0-micron quartz showed slightly higher strengths, and the maximum strength was attained with mixes containing 40 and 45 percent quartz, respectively.

The low strength obtained through use of the silica gel can probably be attributed to the great amount of mixing water required for desired consistency. This was also noted by Butt and Krzheminskii.[4] In this investigation the cement/water ratio was about 0.8 for the 40 percent addition of silica gel; the ratio for the 40 percent addition of the 12.7-micron quartz was 2.4. The cement/water ratio was about 2.8 for a comparable mix containing the 18.6- and 20.0-micron quartz and also for the neat portland cement.

To test the effect of varying certain phases of the standard steam cure (182 C, 24 hr), the cement mix containing 35 percent of 12.7-micron quartz was used as a standard. The phases of the cure which were varied and the resultant strength values are listed in Table 2. When the curing temperature of the steam was decreased from 182 C to 159 C, and then to 126 C, the strength of the resulting cement mixes also decreased. This decrease in strength amounted to only about 12 percent at 159 C, but was about 60 percent at 126 C. The 4-day moist cure at 46 C produced better results than the 24-hr

TABLE 2—EFFECT OF VARYING CONDITIONS OF CURE FOR 35 PERCENT 12.7-MICRON QUARTZ–65 PERCENT CEMENT MIX*

Variation in cure	Strength, lb	Hydrates present
182 C, 24 hr (Standard)	35,100	Platy (T)†, fibrous (T)
159 C, 24 hr	32,100	Platy (T)†, fibrous (T)
126 C, 24 hr	14,000	$Ca(OH)_2$, fibrous (T)
182 C, 48 hr	32,800	Platy, fibrous (T)
182 C, 16 hr	33,700	Platy, fibrous (T)
4-day moist cure (46 C)	24,100	$Ca(OH)_2$, fibrous (T), okenite?
182 C, 24 hr, with no 24-hr setting period	27,400	Platy, fibrous (T)

*Mixtures were allowed to set for 24 hr in air at room temperature prior to steam curing except as noted.
†(T) = tobermorite

steam cure at 126 C for the above cement-quartz mixture, but the strength was still about 31 percent below the results produced by the standard steam cure. Prolonging the curing time at 182 C to 48 hr or decreasing it to 16 hr brought about no increase in strength.

Brown and Carlson[1] allowed their cement samples to set in air at room temperature for 24 hr before steaming, but Menzel[2] found that the age of the concrete when steamed had no effect on the ultimate strength. However, when the 24-hr setting period was eliminated from the standard steam cure in this investigation, the strength decreased appreciably.

Under optimum conditions of steam curing in the autoclave, the strength of selected portland cement-quartz mixes was as much as 25 percent higher than that of the neat portland cement which was given a 4-day moist cure at 46 C.

Modulus of rupture tests showed that for the cement mixture containing 35 percent of the 12.7-micron quartz with the standard steam cure, the strength of the rectangular bars was 2590 psi. The strength of neat portland cement bars moist cured at 46 C was only 1300 psi. These results are based on a sample group of 15 bars for each cement mix. The bars were molded at the same time and tested approximately 2 months later.

RESULTS OF X-RAY DIFFRACTION STUDIES

A systematic x-ray diffraction study was made on the various cement mixes produced in this investigation. Three of the known calcium silicate hydrates, dicalcium silicate alpha-hydrate and both the fibrous and platy forms of the low-lime hydrate generally known as tobermorite, were found in many samples.

By x-ray diffraction, the unhydrated portland cement used in this investigation was found to contain large amounts of beta-dicalcium silicate and tricalcium silicate plus a small amount of tricalcium aluminate. These compounds persisted to varying degrees in some of the hydrated cements. The x-ray pattern exhibited by the material identified as the C_2S alpha-hydrate was almost identical to a pattern listed by Kalousek *et al.*[5] for a 2.0 C/S mixture of $Ca(OH)_2$ and quartz cured hydrothermally for 3 days at 175 C. The pattern for the fibrous form of tobermorite reported here is somewhat

TABLE 3—X-RAY DATA OF SIGNIFICANCE IN IDENTIFYING PLATY TOBERMORITE

Tobermorite (this investigation)		Tobermorite[8] (natural)		CSH-I[7]		Tobermorite[6] (Kalousek)	
I/I_o*	d, A	I/I_o	d, A	I/I_o	d, A	I/I_o	d, A
S	11.4-11.5	VS	11.4	S	11.0	VS	11.2
W	5.5†	MW	5.47	MS	5.6	M	5.4
VS	3.09	S	3.10	VS	3.07	VS	3.07
S	2.99	M	2.99	S	2.97	S	2.96
M	2.82†	MS	2.82	S	2.80	S	2.80
S	1.84	M	1.847	VS	1.83	S	1.838

*Relative intensities of peaks: S, strong; M, medium; W, weak; V, very.
†These lines were apparent only at C/S ratios of 0.9 and below.

similar to the patterns listed by Kalousek[6] for his fibrous hydrates formed by hydrothermal treatment at 175 C from mixtures of $Ca(OH)_2$ and quartz or silicic acid.

The x-ray pattern exhibited by the material identified as the platy form of the hydrate tobermorite is similar to the pattern published by Taylor[7] for CSH-I and by Kalousek[6] for $C_4S_5H_5$. However, the pattern conforms more closely with that given by Claringbull and Hey[8] for the naturally occurring form of tobermorite (Table 3). In this investigation the shape and intensity of the peak corresponding to the 002 spacing and listed as 11.4-5A varied somewhat with the C/S ratio of the starting mixture. At a ratio of 0.9, which produced the maximum strength for the 12.7-micron quartz additions, the intensity and sharpness of this peak was at a maximum (Table 4). As the ratio increased or decreased from this point, the 11.4-5A peak diminished in intensity and sharpness, sometimes forming a doublet. No definite trend in the shift in location of this peak with changing C/S ratio could be discerned. In general, the x-ray diffraction peaks associated with the tobermorite-type hydrates were of low intensity and not well defined.

The pattern exhibited by the fibrous tobermorite phase was characterized mainly by the large irregularly shaped peak corresponding to a spacing of 3.04A. The 7.3 and 1.824A peaks listed by Kalousek[6], if present at all, were of very low intensity. In some cases where both of the tobermorite-type hydrates were known to be present by the existence of both the 3.09 and the 3.04A peaks, there appeared to be a broad double peak corresponding to a spacing of 1.84 and 1.82A. The 2.77A peak listed by Kalousek was probably present but in most cases it was obscured by interference with the 2.78A peak for beta-dicalcium silicate. It was difficult to establish the relative abundance of the fibrous phase, as indicated by the intensity of the 3.04A peak, because of the proximity to the 3.09 and 2.97A peaks for the platy phase. The peaks representative of the alpha-hydrate were much sharper and of higher intensity than the tobermorite peaks.

Neat cement

An x-ray diffraction examination of neat cement, cured 4 days in the production moisture chamber at 46 C, showed that it consisted mainly of $Ca(OH)_2$ and unhydrated dicalcium and tricalcium silicate. There was a small and

somewhat diffuse peak at 3.04A, indicating the possible existence of the fibrous tobermorite phase.

When this neat cement (C/S, 3.04) was cured 24 hr in an autoclave at 182 C, dicalcium silicate alpha-hydrate was formed in moderate amounts. The amount of fibrous tobermorite produced at this temperature was nearly three times that at 46 C, as indicated by the increase in intensity of the 3.04A peak. The amount of $Ca(OH)_2$ was about half that obtained at the lower temperature. The amount of unhydrated cement components was also reduced considerably. The intensity of the 2.18A peak was taken to be the best indicator of the amount of unhydrated cement remaining.

12.7-micron quartz, 182 C cure

When 10 percent of the 12.7-micron quartz was added (C/S, 2.05) and the mixture steam cured, the amount of alpha-hydrate tripled, while the free lime completely disappeared (Table 4). The amount of fibrous tobermorite was considerably decreased. In a similar experiment, Kalousek and Adams[9] found by the use of DTA (differential thermal analysis) that the free lime was completely reacted upon the addition of 9 percent of ground quartz. The maximum amount of the alpha-hydrate was also produced with this composition.

When 20 percent quartz was added (C/S, 1.44), the intensity of the alpha-hydrate peaks was decreased to about one-third of the previous intensity, or about the same as that in neat cement. The amount of fibrous tobermorite present remained about the same as with the 10 percent addition. This 20 percent mixture yielded the first indication of the existence of the platy phase of tobermorite. The 3.09A (platy phase) peak and the 3.04A (fibrous phase) peak were of nearly the same intensity.

Further additions of the 12.7-micron quartz resulted in the disappearance of the alpha-hydrate, but the amount of platy tobermorite increased until a maximum was reached at 35 percent quartz. With the 40 and 50 percent additions, the amount of platy phase decreased somewhat and the 11.4-5A peak was considerably broader. The intensity of the 3.04A peak decreased appreciably with the 50 percent addition but remained nearly the same up to that point.

12.7-micron quartz, variable cure

X-ray patterns were run on the cement mixture containing 35 percent of the 12.7-micron quartz (C/S, 0.9) after varying certain conditions of the standard steam curing cycle as listed in Table 2. Reducing the curing temperature to 159 C decreased the amount of platy tobermorite present and was accompanied by a decrease in strength. After curing at 126 C, the platy phase disappeared and only a small amount of the fibrous phase, along with a fair amount of free lime, was present. Prolonging the curing time at 182 C to 48 hr or reducing it to 16 hr had only a slight effect on the amount of the platy phase produced. Elimination of the 24-hr setting period prior to curing brought about an appreciable decrease in the content of the platy phase.

TABLE 4—STRENGTH AND IMPORTANT CRYSTALLINE COMPONENTS IN STEAM-CURED CEMENT–12.7-MICRON QUARTZ MIXTURES

Mixture, cement: quartz	C/S ratio	Strength, lb	Hydrates present	Relative intensity of 11.5Å platy tobermorite peak
100:0	3.04	10,200	$Ca(OH)_2$, $C_2SH(A)$*, fibrous (T)†	—
90:10	2.05	14,300	$C_2SH(A)$, fibrous (T)	—
80:20	1.44	27,800	$C_2SH(A)$, fibrous, platy (T)	5.5
75:25	1.25	32,000	fibrous, platy (T)	4.5
70:30	1.05	32,200	fibrous, platy (T)	6
65:35	0.90	35,200	fibrous, platy (T)	10
60:40	0.77	31,000	fibrous, platy (T)	6
50:50	0.56	28,400	fibrous, platy (T)	3.5

*Dicalcium silicate alpha-hydrate
†(T) = tobermorite

When this same cement-quartz mixture was cured 4 days in the moisture chamber at 46 C, only a trace of the fibrous phase plus a large amount of free lime could be identified. However, the pattern showed a fairly strong peak corresponding to a spacing of about 8.9A. This peak might indicate the presence of the hydrate okenite, CS_2H_2 (based on x-ray data given by McMurdie and Flint[10]). Bernal[11] has indicated that okenite might be formed at relatively low temperatures.

Other quartz, and silica gel

X-ray patterns were also run on cement mixtures containing the other types of quartz used in this investigation with the standard steam cure (182 C, 24 hr). Patterns for the 18.6-micron quartz mixtures were similar to those corresponding to the same percent addition of 12.7-micron quartz. However, there was a striking difference with the 20.0-micron quartz mixtures. Here, the maximum platy tobermorite peak intensity occurred at the 45 percent addition (C/S, 0.66). At the 35 percent addition of the 20.0-micron quartz, the pattern, except for the size of the quartz peaks, was very similar to that of the 20 percent addition of the 12.7-micron quartz; that is, the mixture contained about equal amounts of both phases of tobermorite along with the C_2S alpha-hydrate. This would indicate that the coarser variety of quartz reacts more slowly than the 12.7-micron quartz, a larger addition to the cement being required to produce the same amount of reaction products. It is difficult to understand why this was not true with the 18.6-micron quartz.

Addition of the silica gel in various proportions to the portland cement produced a very weak cement mixture. An x-ray pattern was run only on the mixture containing the 35 percent addition of this material cured 24 hr at 182 C. The pattern exhibited a strong diffuse peak at 3.04A indicating the presence of a considerable amount of fibrous tobermorite. This was the only hydrate present. Kalousek[6] found that a much longer autoclaving time (3 days) at 175 C was necessary to form the platy phase when using 0.8 ratio mixtures of $Ca(OH)_2$ and silicic acid rather than $Ca(OH)_2$ and ground quartz.

INTERPRETATION OF RESULTS

Formation of dicalcium silicate alpha-hydrate

For mixtures of C/S ratio ranging from 1.44 to 3.04 and cured at 182 C for 24 hr, dicalcium silicate alpha-hydrate appeared. This hydrate could not be produced when a mixture with C/S ratio of 1.44 (not listed in tables) was cured at 126 C for 24 hr. A small amount of the fibrous tobermorite was present however. It appears that the alpha-hydrate is formed only at relatively high temperatures over this composition range. By x-ray diffraction, Kalousek *et al.*[5] have found this hydrate to be present at 175 C in mixtures of $Ca(OH)_2$ and quartz at C/S ratios of 1.8 to 2.4. Heller and Taylor[12] have found the alpha-hydrate, along with $Ca(OH)_2$, to be the products of hydrolysis of pure tricalcium silicate at temperatures between 100 and 200 C. They postulate that the alpha-hydrate is stable over this temperature range in the presence of excess $Ca(OH)_2$, but that it can be no more than a transition product when formed from starting materials of its own C/S ratio or lower.

Using mixtures of portland cement and ground quartz, Kalousek and Adams[9] found by DTA that an addition of 9 percent quartz was sufficient to react with all the free lime at 175 C. Their maximum amount of alpha-hydrate was also produced with this addition. Menzel[2] found the strength to be a minimum at about this composition. Butt *et al.*[13] have also found by DTA that in hydrothermally treated mixtures of quartz and tricalcium silicate, the maximum amount of alpha-hydrate coincides with a minimum of mechanical strength. The results of this investigation substantially confirm these findings.

Formation of transitional hydrates

Kalousek[6] has stated that with 0.8 C/S mixtures of $Ca(OH)_2$ and ground quartz cured at 175 C the lime first reacts with the siliceous material to form the lime-rich phases which may include the C_2S alpha-hydrate. Further hydrothermal treatment then results in a reaction between the residual quartz and the lime-rich phases to form the tobermorite-type hydrates. He postulates that this transition is gradual and involves intermediate transitions through a number of hydrates of progressively lower C/S ratio. The C/S ratios of these intermediate hydrates range from 1.33 to 0.8. Their x-ray diffraction patterns were nearly the same and all contained a large diffuse peak at 3.04A. He found them to be fibrous in nature when viewed under the electron microscope. If the reaction is then carried to completion under these hydrothermal conditions, the resulting product is the platy or stable phase of tobermorite. This material, given the formula $C_4S_5H_5$, was found by Kalousek to be in the form of flat or rolled plates as viewed under the electron microscope.

In this investigation, the existence of any lime-rich phases as transitional hydrates in the 0.9 or 0.77 C/S mixtures could not be determined because there were no autoclaving periods shorter than 16 hr. At C/S ratios higher

than about 2.0, and at the same temperature, 182 C, the C_2S alpha-hydrate appears to be the stable phase.

The role of the fibrous tobermorite at these higher C/S ratios, and at 182 C, is not clear. In all cases where the alpha-hydrate exists, the fibrous phase is also present. Under these conditions the maximum amount of the fibrous phase is produced at a C/S ratio of 3.04 (pure portland cement). With the addition of 10 percent quartz the 3.04A fibrous tobermorite peak diminishes considerably.

With the 20 percent addition of quartz the fibrous phase increases slightly and the platy phase appears. The fibrous phase was found in all mixtures which contained the platy or strength-giving phase of tobermorite. With further additions of quartz the amount of fibrous phase seemed to decrease slightly and the platy phase increased. The fibrous phase did not begin to diminish appreciably until the addition of 50 percent quartz (C/S, 0.56).

At temperatures below about 126 C and at all C/S ratios, the fibrous phase is the only calcium silicate hydrate existing and is always accompanied by some free lime. The one exception might be the 0.9 mixture which was moist cured for 4 days at 46 C. Here, there was some indication of the presence of okenite, CS_2H_2, along with the fibrous phase and free lime. This same mixture, when steam cured at 126 C for 24 hr, developed more of the fibrous phase and less free lime. The moist-cured (46 C) neat portland cement contained only the fibrous phase with a large amount of free lime.

There was no distinct correlation found between the amount of the fibrous phase present and the mechanical strength of the various cements tested.

Formation of platy tobermorite

The results of this investigation show clearly that if cured under the proper conditions of time and temperature, a substance previously identified as the platy phase of the calcium silicate hydrate, tobermorite, is the principal bonding material in an autoclaved concrete product containing portland cement and ground quartz. This platy tobermorite could be produced by using starting mixes of cement and ground quartz ranging in composition from a C/S ratio of 1.44 down to 0.56 and possibly lower, when cured hydrothermally at 182 C for 24 hr. The maximum amount of platy tobermorite was produced in the 12.7-micron quartz mix with a starting C/S ratio of 0.9 and the resulting cement mixture exhibited the greatest strength. Menzel[2] has found that quartz additions between 30 and 35 percent produced the strongest cement mixtures. Butt *et al.*[13] have shown that in hydrothermally treated mixtures of quartz and beta-dicalcium silicate, a maximum amount of their "0.9 hydrate" occurs with the addition of 40 to 50 percent quartz, coinciding also with a maximum in mechanical strength.

The minimum autoclave temperature needed here to produce the platy tobermorite lies between 126 C and 159 C for the 0.9 mixture; a temperature of 182 C resulted in the formation of the greatest amount of the platy tobermorite. At 182 C, curing periods of 16, 24, and 48 hr produced the platy phase

in nearly the same amounts. Kalousek found that when using a 0.8 C/S mixture of $Ca(OH)_2$ and quartz formation of the platy tobermorite was nearly complete in 8 hr at 175 C.

Results here also show that the composition and hydrothermal conditions of time and temperature needed to produce the platy tobermorite will vary with the type and particle size of the siliceous aggregate used.

The most outstanding increase in strength of cement containing the platy phase of tobermorite was shown by the modulus of rupture tests on bars made from the 0.9 C/S mixture of 12.7-micron quartz and portland cement which was steam cured 24 hr at 182 C. Bars containing the platy tobermorite phase had exactly double the strength of the moist-cured neat portland cement bars. Since under transverse loading the initial fracture of the cement bar would occur at the surface under tension, these results indicate that the sheet-like platy tobermorite crystals are much more effective in increasing the tensile or bonding strength of the resulting cement. This increase in tensile strength may not have been completely realized with the metal test units where the cement was subjected to a shearing type of stress.

SUMMARY

1. Because of differences in particle size and reactivity of the various types of silica, optimum strength was not produced at the same C/S ratio of the cement-silica mixtures in each case. The coarser the silica, the lower the C/S ratio to produce optimum strength under given conditions of steam curing.

2. The mechanical strength of steam-cured cement-silica mixtures could be correlated with the amount of the platy calcium silicate hydrate, tobermorite, which was formed.

3. Any steam-cured mix which contained platy tobermorite was as strong or stronger than the moist-cured neat portland cement; any which did not was weaker.

4. Changing conditions of steam curing from optimum (182 C, 16-48 hr) brought about both a decrease of tobermorite and mechanical strength.

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Title No. 54-10

Fatigue and Static Tests of Steel Strand Prestressed Beams of Expanded Shale Concrete and Conventional Concrete*

By GENE M. NORDBY† and WILLIAM J. VENUTI‡

SYNOPSIS

Tests on 27 beams cast from conventional and expanded shale aggregate concrete, prestressed with steel strand, are discussed. Fatigue tests at various load ranges and number of cycles of load repetition were performed on matched beams manufactured from both aggregates. Steel fatigue failures occurred in three specimens while the other 24 beams performed satisfactorily under fatigue loading. The three fatigue failures occurred when the beams were severely cracked during the repetitive loading; this failure was a result of stress concentrations and abrasion between the strands and the concrete. Those specimens undamaged by the fatigue loading were tested statically to failure, and either flexure or bond failures were recorded. The bond failures indicated that embedment length was the governing factor against failure rather than bond stress as computed from conventional equations.

INTRODUCTION

Use of lightweight aggregates, such as expanded shale, has been generally accepted in building construction in recent years. The weight-saving characteristics of lightweight aggregates are often an indisputable advantage; however, problems of shrinkage, creep, and general strength have restrained the practicing engineer from completely accepting it in prestressed concrete. In addition, little is known about the fatigue resistance of prestressed concrete. It seemed that many of the unknown factors about prestressed concrete, such as allowable bond stress, might be particularly critical under fatigue loading. Consequently, this project was undertaken with a two-fold purpose: (1) to study the use of a lightweight aggregate in bonded-type prestressed concrete beams; and (2) to explore the effects of fatigue loading on prestressed concrete beams made with both conventional stone aggregates and expanded shale aggregates. Some knowledge was also gained on bond between steel strand and the two concretes as a by-product of the other tests.

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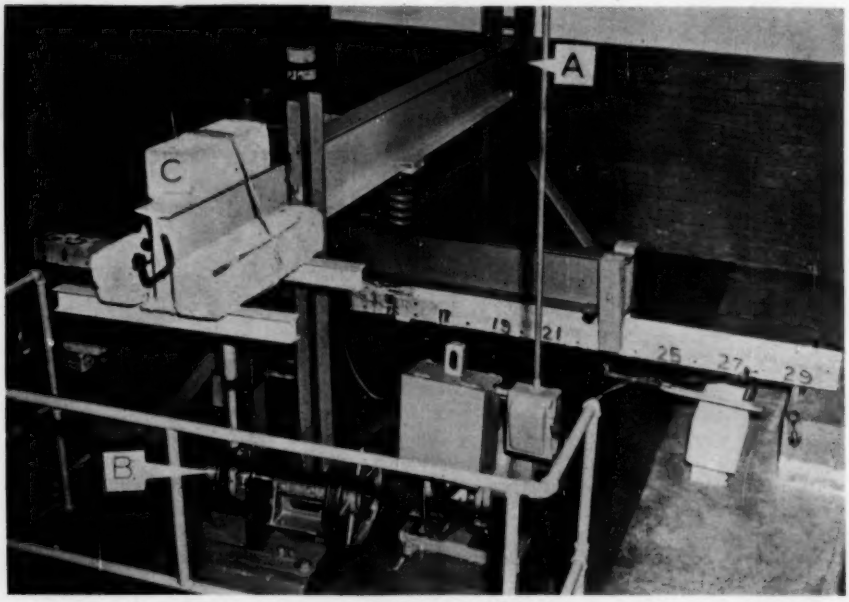

Fig. 1—Fatigue machine

NOTATION

A_s = area of prestressing steel
b = width of beam
d = depth of beam from top of beam to center of gravity of steel tendons
D = nominal diameter of strand
e_s = eccentricity of a particular strand with respect to transformed cross section
f_s = steel stress, in general
I_c' = moment of inertia of transformed cross section
jd = distance between internal compression force and internal tension force of the beam
L_e = embedment length, in general (measured from crack location to end of beam)
n = modular ratio
p = percentage of steel = $A_s/bd \times 100$
p' = percentage of steel based on transformed cross section
u_f = bond stress due to flexure
u_u = total bond stress at failure
V = total shear
$\sum_o$ = sum of the perimeters of strand in a beam based on a periphery of $4\pi D/3$ for each strand

SPECIMENS AND TESTING

Materials

Seven-wire uncoated stress-relieved strands of 5/16-in. and 3/8-in. diameter were used for pretensioning the beams. Tests on the 3/8-in. strand (tested as a unit) indicated ultimate strength of 272,000 psi, modulus of elasticity of 28.75×10^6 psi, and stress at 0.2 percent permanent strain of 240,000 psi. Tests on individual wires which make up the strand yielded slightly higher values. The listed properties were used in all calculations for the tests described herein.

Two aggregates were used in the concrete—conventional stone aggregate and an expanded shale aggregate. The expanded shale is of special interest. The shale in its natural form comes from the Pierre formation in Colorado and is quarried and crushed to size from 0 to

¾ in. Different sizes are separated and calcined separately in a rotary kiln. At approximately 2100 F, the critical temperature, the surface of the individual particles becomes a semi-viscous liquid, and when bloating occurs the viscous surface stretches around the particles much like a rubber balloon to form discrete surface-sealed particles. Manufactured sizes range from sand sizes up to and including ¾ in.

A local stone aggregate similar in gradation to the shale was used for comparison. A cement-water ratio of 0.41 by weight was maintained in the concrete made from both aggregates to facilitate this comparison. An overnight steam cure was used to rapidly bring the concrete to a strength above 4000 psi (most often over 5000 psi) so that the prestress could be released and the beams removed from the prestressing beds, although in some cases prestress was not released until 7 days had elapsed in order to allow time for attaching gages. The average slump of both concretes was ½ in.

Both concretes gave approximately equal strengths for equal water-cement ratios (5500 psi to 6500 psi in 28 days). Comparable shrinkage values (less than 300 μin. per in.) were recorded for both aggregates over a 2 month drying period. Tests revealed that the modulus of elasticity and modulus of rupture were smaller for the shale concrete than for the conventional concrete. The modulus of elasticity was approximately 2.5×10^6 psi for the shale concrete and 3.6×10^6 psi for the stone concrete. The modulus of rupture averaged $0.074 f_c'$ and $0.109 f_c'$ for the shale and stone concretes respectively. The average hardened unit weight was 100 lb per cu ft for the shale concrete and 146 lb per cu ft for the stone concrete.

Fatigue machine

A special fatigue machine (Fig. 1) was used to apply the repeated load to the beams. It consists of a steel walking beam which pivots between two upright supports A anchored in concrete. The other end floats between two uprights. The walking action is provided by an adjustable eccentric B mounted below the floating end. The load is provided by placing concrete billets C on the floating end of the beam. The machine provides constant load rather than constant deflection because the walking beam was able to follow the test beam as its maximum deflection gradually increased due to the repeated load. The loading rate was 90 cycles per min.

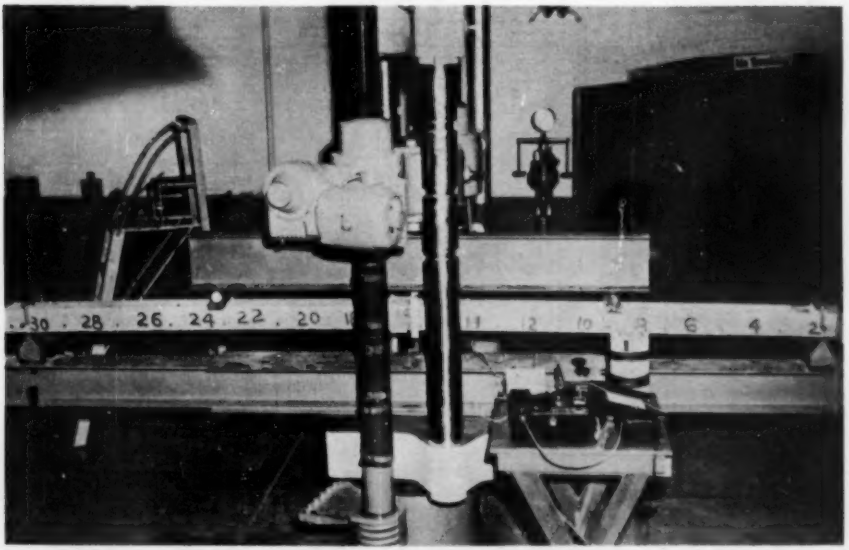

Fig. 2—Static testing apparatus

Static tests

Static loading of the beams was accomplished in a large hydraulic testing machine by placing a large base beam on the platen of the testing machine as shown in Fig. 2. Deflections were measured at the center line and dial gages were attached to the ends of the beams to record the slip of the strand. Electric strain gages were attached both externally on the concrete surface and internally on the surface of the steel strand for strain measurement in both the static and dynamic tests. The loading spans are illustrated in Fig. 3 and are referred to in Table 1 for each beam.

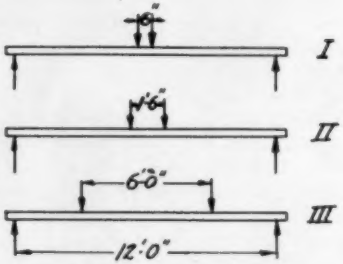

Fig. 3—Loading spans for static and dynamic tests

Description of beam specimens

Four cross sections of beams as shown in Fig. 4 were tested. Fatigue tests were performed on all sections except Beam D, upon which only static tests were performed.

Beams A, B, and C were all 4½ in. deep and 6 in. wide. Beam A was prestressed with three 5/16-in. diameter strands (A_s = 0.0562 sq in. per strand). In all of the beams reported the steel was given an initial tension of 175,000 psi. After prestress losses, the stress was reduced to approximately 155,000 psi. More precise values are listed in Table 1 for each beam. All of the beams with a similar cross section in a series were cast in line in the casting bed so that all were subjected to the same prestressing force. Two #3 reinforcing bars were placed in the top of the beams to take small tensile stresses.

Beam B was identical to Beam A except that two ⅜-in. diameter strands were used instead of three 5/16-in. strands. The prestress force supplied by either two ⅜-in. strands or three

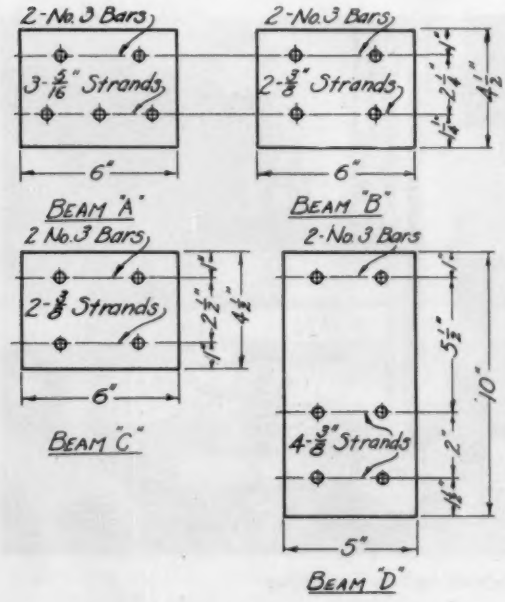

Fig. 4—Beam cross sections

TABLE 1—SUMMARY OF STATIC TESTS

Beam No.	Cross section (Fig. 4)	Load type (Fig. 3)	f_c' (at test)	Concrete prestress, psi		Steel prestress at test, psi	Steel†† stress at failure, psi	Cracking moment, in.-kips	Ultimate moment, in.-kips	Bond at ultimate, psi, Eq. (2)	Type of failure
				Top	Bottom						
Series 1—Conventional concrete beams											
1A	A	II	6660	− 49	1891	160,635	246,070	55.4	114.1	58	Concrete crushing‡
1B	A	II	6660	− 49	1891	160,635	238,290	57.9	114.1	53	Concrete crushing
2A	A	II	6660	− 44	1856	158,065	239,890	—*	118.1	56	Concrete crushing
2B	A	II	6660	− 44	1856	158,065	239,960	60.5	114.9	54	Concrete crushing
3A	A	III	6660	− 44	1856	158,065	233,915	57.6	105.3	86	Concrete crushing
3B	A	III	6660	− 44	1856	158,065	229,910	57.2	103.5	91	Concrete crushing
4A	B	II	6040	− 68	1794	157,320	251,720	61.1	118.1	94	Concrete crushing
4B	B	II	6040	− 68	1794	157,320	275,160	—*	129.1	118	Concrete crushing
5A	B	II	6040	− 66	1784	156,480	251,720	—*	118.1	95	Concrete crushing
5B	B	II	6040	− 66	1782	156,390	258,970	—*	121.2	103	Concrete crushing
6A	B	II	6040	− 66	1784	156,480	—	—*	76.9**	—*	Steel fatigue
6B	B	II	6040	− 66	1786	156,560	—	—*	76.9**	—*	Steel fatigue
Series 2—Shale and conventional concrete beams											
P1	B	II	6400	−121	1737	154,190	236,600	—*	113.4	68	Concrete crushing
P2	B	II	6400	−121	1737	154,190	239,900	—*	114.9	71	Concrete crushing
P3	B	II	6780	−115	1697	150,990	246,400	—*	118.1	78	Concrete crushing
S4	C	III	5840	−360	1910	148,600	204,900†	55.8	81.0	69	Bond§
G4	C	III	5430	−300	1950	151,600	217,700†	47.5	83.7	69	Bond§
S5	C	III	5480	−350	1890	147,300	—*	—*	79.8	—*	Bond§
G5	C	III	5430	−300	1950	151,300	—*	—*	83.7	—*	Bond§
S6	C	II	5600	−370	1940	150,700	—*	60.7	76.9**	—*	Steel fatigue
G6	C	II	6345	−310	1980	153,600	244,900	59.2	123.8	74	Concrete crushing
Series 3—Shale and conventional concrete beams											
S3	D	I	5820	−310	2080	152,100	214,600†	310.5	427.8	46.5	Bond
G3	D	I	5390	−310	2120	152,500	272,000†	308.7	552.0	93	Concrete crushing
S1	D	II	5535	−340	2150	157,100	206,000†	275.6	406.3	41	Bond
G1	D	II	4956	−320	2180	156,900	216,400†	264.6	346.5	39	Bond
S2	D	III	5840	−340	2120	154,900	190,400†	260.1	378.0	55	Bond
G2	D	III	5470	−320	2160	154,900	174,300†	229.5	306.0	26	Bond

NOTE—S indicates shale concrete; G indicates gravel (conventional) concrete. P1, P2, and P3 were shale concrete.
*Not observed
†Recorded by gage
‡Concrete crushing refers to concrete failure as result of steel yielding.
§Bond failure refers to initial slip of strand resulting in secondary shear failure.
**Fatigue moment
††For Series 3, bottom strand is listed. Stresses are at center line of beam.

5/16-in. strands was very nearly the same, which facilitated comparison of Beams A, B, and C. Beam C was identical to Beam B except that the strands were placed only 1 in. from the bottom of the beam instead of 1¼ in. as in Beam B. The area of steel in Beams B and C differed slightly since a change in manufacturing during the testing period changed the area of a ⅜-in. strand from 0.0826 sq in. to 0.0799 sq in.

Only static tests were performed on Beam D which was 5 in. wide and 10 in. deep. Four ⅜-in. strands in two layers were used to prestress it. This section was tested mainly to examine dual layers of strands, bond theory, and embedment length under various conditions of loading.

Description of tests

Tables 1, 2, and 3 summarize static and fatigue tests, the resulting stress reversals, and other information. Design load was based on the moment necessary to produce zero stress in the lower fibers of the beam. This load was calculated before test and was based upon estimated prestress losses. The design load was approximately 27 to 29 percent of the ultimate load for those beams failing in flexure. The cracking load in these same cases was approximately 50 percent of the ultimate load. Refer to the tables throughout the following description for additional details.

Series 1—Two beam cross sections were used in the first phase of the investigation. Six beams of each of the cross sections A and B were cast with conventional concrete only. The beams were matched in pairs as to finish, camber, and outward appearance so that the physical properties would be nearly the same. One beam of each set was loaded with the fatigue load for a predetermined number of cycles. If it did not fail under the fatigue loading, it was tested to destruction in a static test. The mate, or comparison beam, was also tested to destruction under static load and the results were compared. In some cases the comparison beam was also given a fatigue loading.

The static tests usually demonstrated a typical failure caused by exceeding the yield strength of the steel. This elongation of the steel forces the neutral axis toward the top of the beam, resulting in crushing of the concrete as shown in Fig. 5. Upon release of the ultimate load, the beam again made a significant recovery and could still support about 60 percent of the ultimate load.

Typical deflections of two pairs of a fatigued beam and an unfatigued beam are shown in Fig. 6. In each case the fatigued beam is stiffer than the unfatigued. The upper set of curves (3A and 3B) illustrates the most pronounced stiffening effect after the most severe repetitive load (73 percent ultimate load) while the lower set (1A and 1B) illustrates only a slight

TABLE 2—SUMMARY OF FATIGUE TESTS

Beam No.	Repetitions of loading	Fatigue load: Percent of ultimate load	Fatigue load: Percent of design load	Fatigue moment, ft-kips	Fatigue shear, lb	Steel, range of stress, psi	Concrete, range of stress, psi	Bond due to flexure, range of stress, psi
1A	1,014,100	29.4	100	33.7	535	5000	1560*	3.4
2A	1,000,000	65.0	240	76.9	1220	21,340	6208*	14.7
3A	2,000,000	73.0	240	76.9	2135	11,485	—†	13.6
4A	9,653,000	27.6	100	32.7	520	4540	1530*	3.8
5A	1,108,700	41.5	150	49.1	780	6800	2295	5.7
	3,000,000							
5B	1,100,000	54.0	200	65.5	1040	—†	—†	—†
6A	136,000	67.0§	240	76.9	1220	23,890	—†	20.7
6B	186,000	67.0§	240	76.9	1220	23,890	—†	20.7
P2	1,100,000	28.5	100	32.7	520	6600	1470	3.9
P3	2,168,000	55.5	200	65.5	1040	—†	—†	—†
S5	1,000,000	28.0§	100	36.0	1000	8900*	1550*	1.3
	1,000,000‡	30.0§		39.2	1265	25,700*	2000*	20.7
G5	1,000,000	28.0§	100	36.0	1000	7300*	1620	1.3
	1,000,000‡	22.0§		28.0	775			—†
S6	842,000	62.0	240	76.9	1220	29,400	2110*	24.0
G6	2,000,000	62.0	240	76.9	1220	24,400	2360*	23.4

*Based on actual strain measurements.
†Not observed.
‡Applied after static test to cracking load.
§Based on average failure moments.

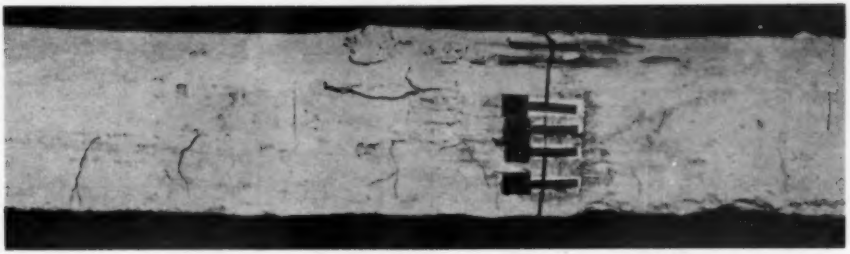

Fig. 5—Typical flexural failure

stiffening effect after repetitive load at the design load (29.4 percent ultimate load). The more severe the fatigue loading, the stiffer the beam became. In fact, the beams seemed to be actually improved in most cases by the fatigue action with the exception of Beams 4A and 4B. Beam 4A endured nearly 10 million cycles of the design load (27.6 percent of ultimate load) without ill effect when compared to others in the same series.

Two of the beams in this series failed by fatigue of the steel strand. Beam 6A was placed in the fatigue machine and 2.4 times the design load (67 percent of the ultimate load) was applied and failure occurred after 136,000 cycles of load. Since a comparative static test could not be made with Beam 6A, it, too, was tested under the same fatigue load and failed after 186,000 cycles of load. These fatigue failures are discussed in a later paragraph. No other fatigue damage was noted in this series nor was there any evidence of slip or bond failure.

Series 2—This series of tests was designed to compare beams made of expanded shale and conventional aggregates. First, three expanded shale beams of cross section B were cast as pilot tests. P1 was tested with a static load, P2 with one million cycles of the design load (28.5 percent ultimate load) and then statically to failure, and P3 with two million cycles of twice the design load (57.0 percent ultimate load) and then statically to failure. The deflection curves are illustrated in Fig. 7 for the final static tests and the final results are in Tables 1 and 2. Note that no increased stiffness is shown for the heavily fatigued beam as was the case for the previously described Series 1 beams of conventional concrete.

Since the pilot beams were successful, six more beams were cast, three of which were conventional concrete for direct comparison with three additional expanded shale beams. The water-cement ratio and the ultimate compressive strength were approximately equal for both types of concrete used in these tests. First, a matched pair of beams, one conventional concrete and one shale concrete (G4 and S4, G for gravel and S for shale), was loaded statically to failure at the quarter points. Center line deflection curves for these beams are shown in

TABLE 3—MEASURED STEEL STRESS RISE AND BOND STRESSES AT SLIP AND FAILURE

Beam No.	Stress rise at first slip, psi				Stress rise at ultimate load, psi		Bond at first slip, psi	Bond at ultimate psi, Eq. (3)
	Lower strand		Upper strand					
	No. 1	No. 2	No. 3	No. 4	Lower	Upper		
S1	NG	NG	3020*	8000	46,900**	28,600	3.8§	167
G1	11,780*	11,780*	NS	4025*	59,500	16,700	4.7§	175
S2	18,250	18,250	2703*	6320	35,500	20,270	6.6§	330‡‡
G2	13,650	13,650	6470	6470	19,400	10,400	—‡	280‡‡
S3	NS	62,530	25,580	15,525	62,530	25,580	—‡	158
G3	NS	NS	NS	NS	119,500	80,643	—‡	199
S4	33,640	33,640	—	—	66,100	—	—‡	264‡‡
G4	11,070	11,070	—	—	56,350	—	—‡	272‡‡
S5	—†	—†	—	—	75,890††	—	—‡	206
G5	—†	—†	—	—	71,730††	—	—‡	—†

NS = No slip
NG = No gage
*Slipped before crack.
†Not observed.
‡Crack depth not recorded for Eq. (2).
§Eq. (1).
**Computed.
††After slip in fatigue test.
‡‡From computed stress at quarter point.

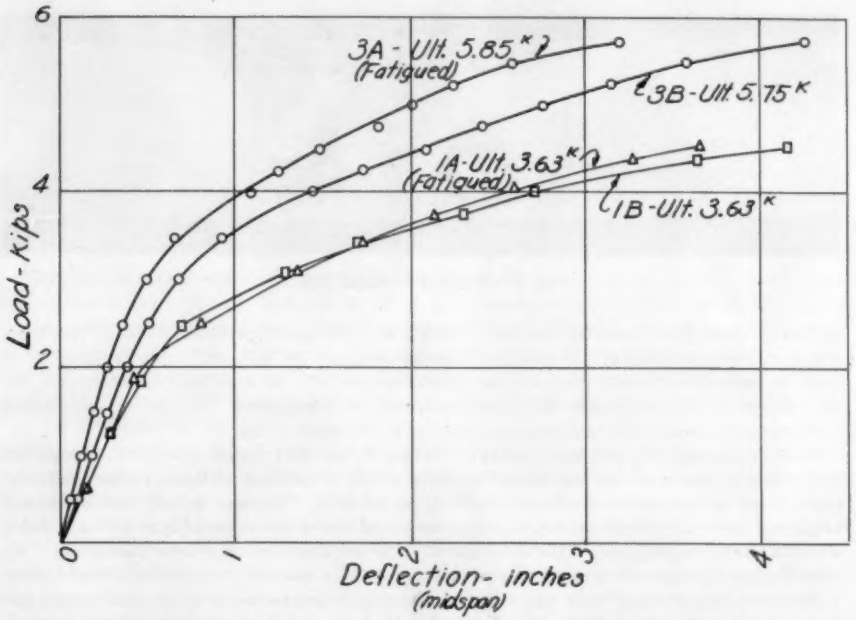

Fig. 6—Typical deflection curves for pairs of fatigued and unfatigued beams

Fig. 8. Note the greater deflections obtained for the shale beam as compared with the conventional beam because of the lower modulus of elasticity of the concrete. The cracking loads and ultimate loads for both the shale and stone concrete beams were almost identical (Table 1).

Another set of beams (S5 and G5) was individually loaded by the fatigue machine to the design load at the quarter points of the beam for one million cycles. No damage was in evidence at this point. They were then removed and loaded to 80 percent of the ultimate load in a static test. Severe cracking was evident throughout the center portion of the beam and slip of strand occurred in both beams. They were again placed in the fatigue machine and the test was continued at a load sufficient to open cracks for one million more cycles. It was apparent that complete loss of prestress was prevented by the spiral shape of the strand. It is important to note that the prestress was retained after slip had occurred and during the extensive fatigue loading. The cracks closed tightly after each removal of load and could not be detected, *even* with a magnifying glass.

The last two beams (S6 and G6) were loaded at 2.5 times the design load (62 percent ultimate load) in the fatigue machine. The shale beam exhibited steel fatigue after 842,000 cycles of load, but no such failure occurred in the conventional beam, even after two million cycles, and it was finally loaded to failure in a static test and failed in a manner similar to that illustrated in Fig. 5.

Series 3—In this phase of testing, only static tests were performed upon six beams of cross section D (Fig. 4). The beams were again matched in pairs of one conventional and one shale concrete beam. Each pair was loaded to failure by two symmetrical loads located near the center, at the quarter points, and at an intermediate point as illustrated in Fig. 3.

The failures were almost identical in character in each case as illustrated in Fig. 9. They appeared to be shear failures, but in reality were caused by bond failure. A comparison of the deflection curves of a typical pair of beams from this series is shown in Fig. 15. These failures are further described in a later paragraph; however, such failures are erratic as will be indicated by comparison of the ultimate moments recorded in Table 1 for the Series 3 beams.

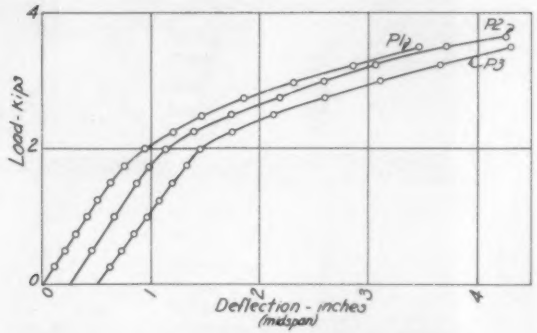

Fig. 7—Deflection curves for preliminary shale beams

Shear specimens—The ends of the beams tested in Series 1 and those of the preliminary tests P1, P2, and P3 of Series 2 were undamaged after failure so they were further tested on shorter spans. The specimens were loaded at the center of a 1-, 2-, or 3-ft span with a ½-ft overhang. Dial gages recorded the slip of strand. In 17 of 28 tests such strand movement was recorded before the specimen was cracked. Deflection was also recorded at the center of the beam.

A typical deflection curve is shown in Fig. 10. All the curves were essentially of the same shape consisting of a straight line portion and a sharp break in the curve when the crack occurred. From this point, the curves sometimes continued smoothly as the deflection increased and other times in abrupt spurts as the steel alternately took hold and was released by friction. In this particular curve one of the strands slipped before the crack occurred. Although a detailed tabulation is not given here for these tests the average bond stress at failure for each series is plotted in Fig. 17 described later.

DISCUSSION OF RESULTS

Prestress losses

A careful record of the elastic losses and the combined shrinkage and creep losses was kept for the beams of Series 2 and 3. This information was used in computing the state of stress in all beams at the time of test. The elastic losses measured by SR-4 gages on the strand were only 5.4 percent on the average below the computed values based on the modulus of elasticity as obtained from the cylinder tests. The creep was measured by the use of external metal buttons and a mechanical extensometer over a period of 90

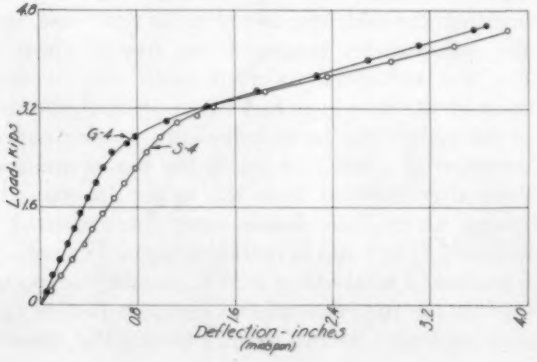

Fig. 8—Comparison of deflection curves of matched shale and conventional concrete beams

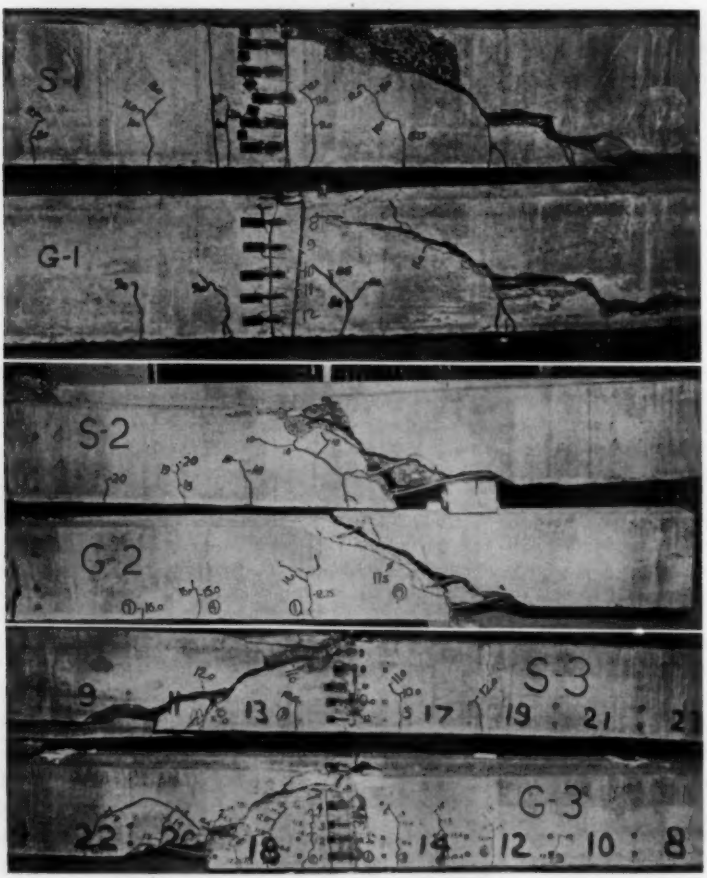

Fig. 9—Beam failures, Series 3

days or more. The results are recorded in Fig. 11 for two pairs of beams. In one case (S3 and G3) conventional concrete beams had a greater creep loss than the shale beams and in the other case, the reverse was true. Although the extensometer measured loss due to creep and shrinkage, most of this loss was attributed to creep, since, due to climatic conditions and curing, most of the shrinkage had taken place at the time of release of the prestress in the beds. The irregularities in the early part of the curve were due to the cessation of a wet cure which the beams underwent in the laboratory for 28 days after removal from the beds. In computing the creep losses for the beams where these losses were not measured, the Caquot equation[1] was assumed to be valid in extrapolating the values. On the basis of the equation, a maximum total loss of 0.18 f_{si} should be expected for the shale concrete and 0.16 f_{si} for the conventional concrete (where f_{si} is steel stress due to initial prestressing). At the time of testing the losses ranged between 0.09 f_{si} and

0.16 f_{si} depending upon the age of the specimen at the test time. The prestress at the time of the test is recorded in Table 1.

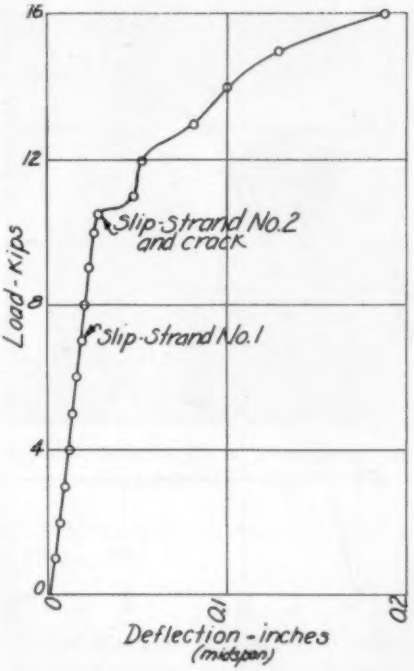

Fig. 10—Typical shear specimen deflection curve

Steel fatigue failures

Three beams (6A, 6B, and S6) exhibited steel fatigue failures. The same external load was applied to each of the beams which induced steel stresses ranging between 21,000 psi and 29,000 psi. In the fatigue machine the beams were all loaded at approximately 67 percent of the ultimate strength of the beam as previously described.

All of these beams were severely cracked during this loading and it is believed that the cracking and rubbing of the concrete against the steel affected the endurance limit of the beam. Several wire coupons were taken from the ruptured strand and a precise tension test was run on each. Fig. 12 indicates the difference in the results of these tests. Curve A illustrates the stress-strain diagram for a specimen taken from the end of Beam 6A where there had been little change in stress. With this sample, a normal-stress diagram was obtained with a yield strength (0.2 percent offset) at 270,000 psi and with

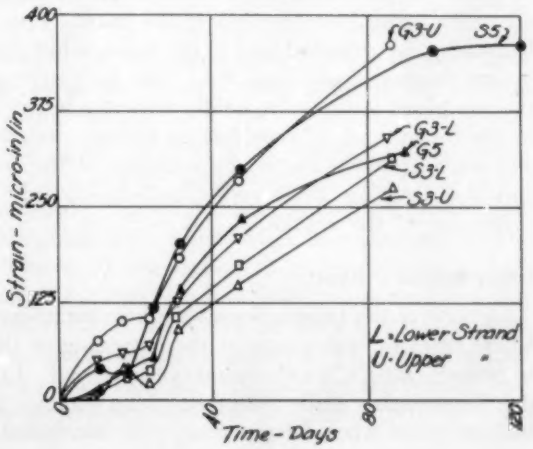

Fig. 11—Creep losses in beams at steel line versus time

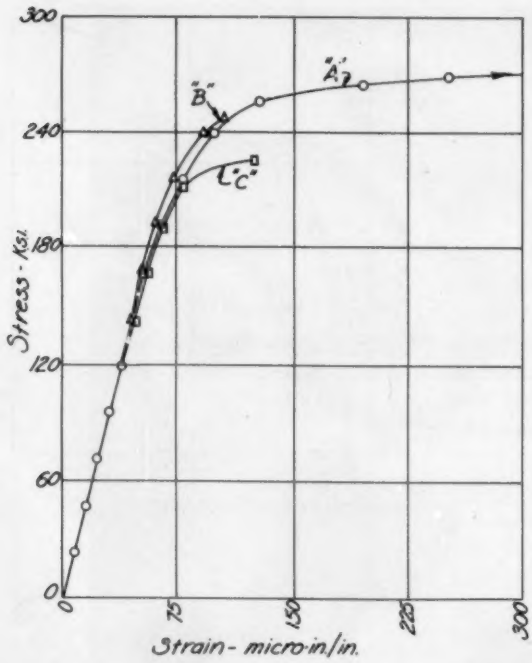

Fig. 12—Stress-strain diagrams for wire coupons from fatigued beam

considerable elongation before failure. Curves B and C represent samples taken adjacent to the fatigue failure. The wire as a whole lacks ductility, and the strength has been impaired, probably due to fatigue damage in adjacent portions of the wire. This effect was noted in the wire from all heavily fatigued beams. Fig. 13 illustrates the type of failures that were noted in these tests. Specimen A (letters match curves in Fig. 12) is a ductile cup-cone break of a specimen taken from the end of the beam. Specimens B and C show brittle fractures after repeated load in the beam, while Specimen D shows a suspected fatigue fracture from Beam 6A. The fatigue progression was probably across the top plane of the wire, finally resulting in failure across the diagonal plane in the foreground. These fatigue failures occurred only in the $\frac{3}{8}$-in. strand and were probably caused by cracking of the concrete under severe loading and the resulting stress concentrations and abrasion during the repetitious reopening of the cracks.

Other fatigue actions

In none of the fatigue tests was there significant loss in prestress. In cases where cracking was severe at the beginning of the test, such cracking ceased to progress after a few thousand cycles of load. In cases where slip was evident due to excessive static loads, such slip did not progress significantly in one million cycles when fatigue tests were continued. Hanson[5] reported fatigue

tests on four beams using both clean and rusted 0.196-in. wire in which bond fatigue failure occurred in all specimens at less than 700,000 cycles (only 7200 cycles in one case) at about 75 percent of the ultimate load. Nordby and Tulin[8] also have reported impairment of beam strength similar to Hanson's after one million cycles at 33 percent of the ultimate load of an unfatigued beam. Strands of the present test seemed highly superior in bond, compared with these other tests.

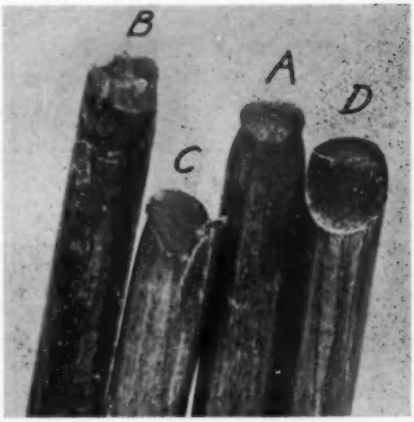

Fig. 13—Failures of wire subjected to fatigue action in beam

There was no visible fatigue damage to the concrete in any of the tests; however, the stresses induced were below the accepted endurance limits for concrete. In one case, a stress of over 6000 psi was induced for one million cycles with no apparent injury (Beam 2A).

Bond failures

Nine beams of Series 2 and 3 failed in bond. This type failure was detected at an early stage by the 0.0001-in. dials attached to the end of the beams to record slip. The stress rise above the existing prestress as recorded by SR-4 gages at the time of initial slip is recorded in Table 3. Such slip occurred at stress increases in the steel as low as 3020 psi and sometimes at a considerable interval before a crack was noticed in the beam.

A typical test proceeded as follows: After loading to about 30 to 40 percent of the ultimate load, the end dials sometimes indicated a slip of 0.0002 to 0.0003 in. had taken place on one or several of the strands. No additional strands slipped until after the beam had cracked. This precrack slip was recorded in three beams of Series 3, the top strand being the first to slip. This is of special interest since the calculated flexural bond stress is less for the upper strand than for the lower strand.

An example of slip action as recorded by SR-4 gages mounted on the strand at the beam center line is illustrated in Fig. 14 for Beam S1. At a load of 4.0 kips a slip of 0.001 in. was recorded. Note the slight jog in the strand stress-increase curve (Fig. 14) at this point, after which stress increases occurred in a linear fashion with load increases. A jog was noted in the deflection curve of Beam S1 (Fig. 15) at a load of 3.0 kips also just prior to the recorded slip. After crack, the load was released and a permanent stress loss of about 7500 psi was noted. Upon reloading, the stress again increased in a normal fashion until complete failure when all strands slipped. Apparently the friction and wedging action of the spiral strand retained the prestress effectively until failure.

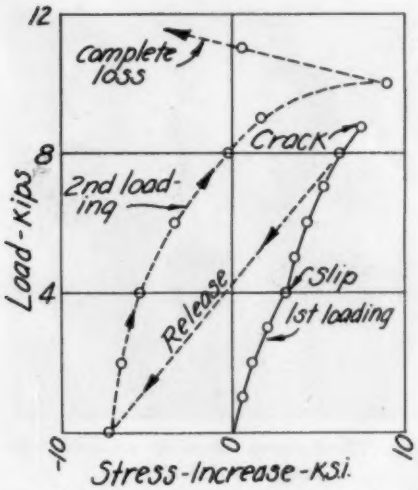

Fig. 14—Steel stress increase versus load during slip action

The first crack appeared as a vertical flexure crack under one of the loads. Soon after this crack occurred the other strands began to slip at periodic intervals, marked by clicks and "talking" from the interior center portion of the beam as a wave of bond stress progressed internally toward the end of the beam. All strands increased their slip to between 0.030 and 0.01 in. during this period. Note in Fig. 14 that slips of these magnitudes result in very little release in prestress and the strain gages continued to record increases in stress as the load increased. A diagonal tension crack started to develop at this stage which joined the original flexure crack in the eventually shattered region. The load could be released and almost complete recovery of deflection could be expected at any time prior to failure.

Any time after the slip reached 0.01 in., failure took place due to shear. This was due to penetration of the crack, the accompanying reduction in prestress across an element in the cracked region and its increased vulnerability to shear failure. Such failures were very erratic and unpredictable.

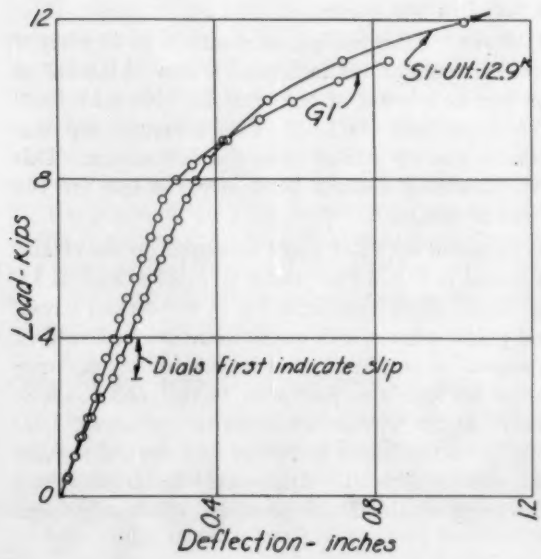

Fig. 15—Typical deflection curves for pairs of beams from Series 3

It may be noted in Table 1 that the ultimate moment resisted by the shale beam was sometimes greater than that endured by the companion stone concrete beam. Only one beam (G3) developed the ultimate strength of the strand before failure, and its shale mate beam came the closest of the shale beams to developing this stress.

When the beams were broken open after the test, (except G2) it was evident that there was no adhesive bond between the concrete and the strand. The grooves always appeared polished due to the movement of the strand relative to the concrete when slip measurable with the end dials had occurred. This polish was sometimes an aid in verifying where slip had occurred. Fig. 16 shows a beam split open after failure. Note the sharply defined protuberances and thread-like pattern still intact after slip had taken place.

Bond stress

The theory of anchorage bond and also the failure mechanism during bond failure have been adequately covered elsewhere.[2] For purposes of this discussion, bond stresses are divided into two parts: (a) the anchorage bond that retains the prestresses in the strand; and (b) the bond due to flexure. It is the latter that is important from a fatigue standpoint. Two expressions are used to describe this bond for the conditions of the precrack and cracked sections. The precrack flexural bond condition is described by

$$u_f = \frac{n\epsilon_s A_s V}{\sum_o I_c'} \quad \ldots\ldots\ldots\

that a modification of the ordinary bond equation for conventional reinforced concrete would be approximately correct as follows:

$$u_f = \frac{V}{\sum_o jd} \times \frac{\text{increase in tensile stress}}{\text{prestress} + \text{increase in tensile stress}} \quad \ldots\ldots\ldots\ldots (2)$$

Since the distance jd cannot be computed, it was found by actual measurement of the cracks on the side of the beams during the tests, or, if this was not observed, a value was assumed consistent with companion tests.

Both Eq. (1) and (2) were used extensively to compute bond stresses at various stages of load. In some cases, when the beam was at an intermediate load such that the crack had not penetrated sufficiently for accurate measurement or had not penetrated both layers of strand, neither equation was valid nor used. In other cases when the strand had already slipped and lost part of its prestress, neither equation was correct. Bond was computed at several critical stages; typical values are indicated in Tables 1, 2, and 3 at various stages of load.

After study of the various bond values in the tables one fact is apparent—the values are extremely low by familiar standards, i.e., when comparing them with values commonly used for reinforced concrete. Apparently, designing with a bond stress limitation will not insure against failure. Bond stress at slip ranged from approximately 3.8 psi upward. In general, it seemed that bond stress was not related to slip, consequently, no limiting bond stresses are recommended for design. It appeared rather that an embedment between the end of the beam and possible crack positions should be specified in order to develop the ultimate strength of the strand at failure. Referring specifically to Series 3, the only beam in this group which developed the ultimate strength of the strand was G3 which has an embedment of 6 ft. This beam did not register slip at any time.

Embedment length

There is no adequate published theory which can be used to insure against bond failure at the present. Bond failure may depend upon a number of factors including size, strength, pretension and surface of the strand, as well as spacing, number, and cover of strands. The type of load also has an effect.

Fig. 17 shows a portion of a beam between the first crack and the end of the beam as it occurred in the beams reported herein. As long as this end remained uncracked the strand behaved as though being pulled from a conventional pullout specimen. If the ordinary bond run-off equation, $u = f_s D/4L$, is modified for the shape of strand, the equation becomes:

$$u_u = \frac{3 f_s A_s}{4\pi D L_e} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

This equation may be used to calculate the average bond stress at the time of failure if the stress in the strand is known. Fig. 17 also shows average bond stresses, so calculated, plotted against embedment length for the various

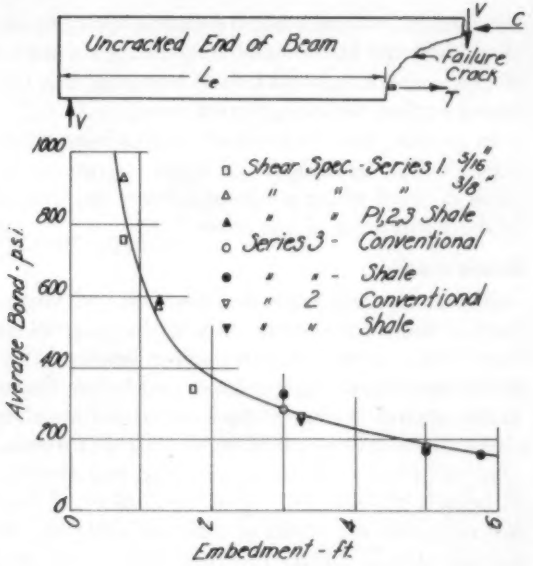

Fig. 17—Average bond stress developed at failure versus embedment length

test beams which failed in bond. The values plotted to the right on the graph were based on actual measured strains in the steel for those beams loaded near the center line (Loads I and II) or the calculated stress at the quarter point for beams loaded at the quarter point (Load III). The center-line stresses based on recorded strains are listed in Table 1, but the calculated values at the quarter points are not shown. The points to the left were based on the calculated steel stress at the cracking load for the shear specimens previously described. The average bond stress decreases rapidly with increased length of embedment and apparently reaches a nearly asymptotic value for embedments of 6 ft or more.

These tests indicated that an embedment length of 6 ft would be required to develop the ultimate strength of the 3/8-in. strand in concrete with $f_c' = 6000$ psi while only a 3-ft embedment would be required for the 5/16-in. strand in the same concrete. An even greater length for the shale concrete—7 or 8 ft—would be necessary. Of course, if these bond stresses are extended to the larger sizes of strand, embedment length needed to protect against ultimate failure becomes quite large. Further research into the effect of cover, spacing, and surface conditions may give more precise limits of use of these strands.

Other recent tests have also indicated the importance of embedment length. Thorsen[3] has reported a slab test which indicated an embedment greater than 4 ft 2 in. would be required for a 5/16-in. strand in a concrete with $f_c' = 4500$ psi. Echols[4] reports that for a concrete with $f_c' = 6600$ psi a 3-ft embedment is sufficient to develop the full strength of a 5/16-in. strand, but that weaker concrete would not develop enough bond to do this in a 3-ft embedment. Janney[2] too reports a bond failure at a stress of 178,000 psi

for 5/16-in. strand with a concrete strength of 4500 psi, and embedment of 39 in. Others[6] have reported results of six beam tests with embedment lengths of 42 in. where no bond failure was recorded, but in this case the strand had a rusted surface which increased bond.

In general, the observations on the beams tested seemed to conform to and support the hypothesis of bond advanced by Janney[2] which describes a wave of bond which progresses down the wire or strand from a crack formed in the midsection of the beam.

Elastic theory

The stresses in both the concrete and steel were well represented by the elastic theory for the precrack loads. Fig. 18 shows the strand stress increase versus load curves for two pairs of beams. One pair (S4 and G4) was loaded to failure without fatigue loads and before the strand had slipped. The stress in the strand is closely represented before a crack occurs by the theoretical stress (dashed line) based on the flexure formula and the modulus of elasticity obtained from a cylinder test. Slip had already taken place in the other pair of beams (S5 and G5) prior to final static loading. The elastic theory did not represent the stress in this case since the steel and concrete did not have mutual strains during precrack loads. All strands had slipped at failure in the four beams shown.

In the beams containing two layers of strand, the steel tended to act as a unit. The stress in the upper layer exceeded the theoretical stress while the stress in the lower layer was less than the theoretical stress. The average stress in the two layers coincided with the theoretical average stress during precrack loads.

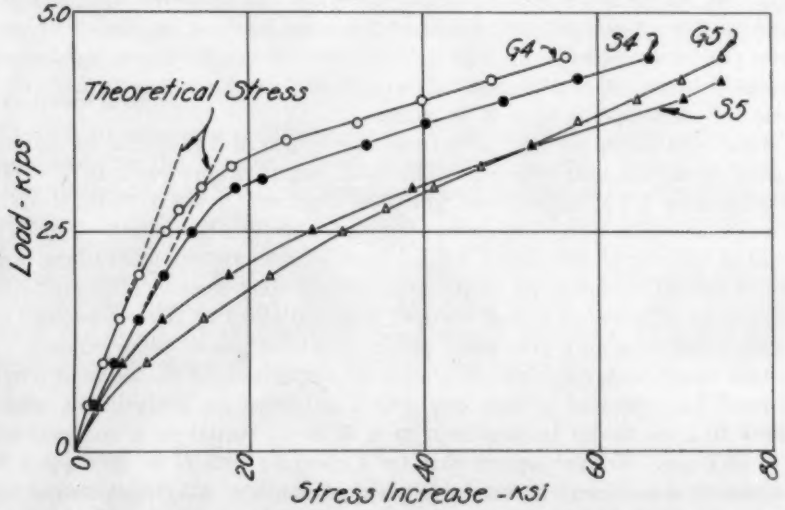

Fig. 18—Typical stress increase versus load curves for beams with bonded and slipped strands

Concrete stresses were not as well represented by the theory. Representation was fair in the lower and middle regions of the beam, but the stresses in the upper portion were erratic and distorted probably because of the two #3 bars in that region. The fact that these beams coincided with current ultimate load theory has been presented elsewhere.[10]

CONCLUSIONS

1. Expanded shale aggregates of the type tested make acceptable concrete for use in prestressed concrete structures. The strength was equal to that of the conventional concrete for equal water-cement ratios. Shrinkage and creep losses encountered were well below accepted maximum values for both the concretes used. The lower modulus of elasticity of the expanded shale concrete was its only deficiency as compared with regular aggregates.

2. The steel strand seemed superior to smooth wire of equal area for use in bonded type design since it had high mechanical bond. This mechanical bond was great enough to prevent complete loss of prestress and failure even though initial slip had occurred.

3. It was impossible to obtain fatigue failures of the beams when they were in the uncracked condition within 10 million cycles of the design load. In fact, it was rare that a fatigue failure occurred even in the cracked condition.

4. Cracks should not be allowed in beams in which repeated overloads occur, since those fatigue failures that did occur appeared to be directly related to cracking. Slip also seemed to be dependent upon cracking to a great extent, although slip sometimes occurred before the crack had occurred.

5. Slip of strand occurred at low bond stress intensities as compu ed by conventional equations. It does not appear that designing with a bond stress limitation will insure against failure; consequently, no limiting bond stresses are recommended. It appeared rather that an embedment between the end of the beam and possible crack positions should be specified, coupled with an allowable stress increase in the prestressing strand and the ultimate concrete strength. Based on the extrapolation of the data for the 5/16-in. and $\frac{3}{8}$-in. diameter strands, it appeared that an anchorage length limitation of a rather severe nature may be required to develop the ultimate strength for strands greater than $\frac{3}{8}$-in. diameter.

6. Elastic losses may be computed by the elastic theory. A fixed strain or shortening may be used for the shrinkage which probably will not exceed 300 μin. per in. Creep losses may be taken as equal to the elastic losses, although further research may indicate a more precise method.

7. The elastic theory is applicable to the concrete and steel stresses when the beam is uncracked for both aggregates.

ACKNOWLEDGMENTS

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Discussion of this paper should reach ACI headquarters in triplicate by Nov. 1, 1957, for publication in the March 1958 JOURNAL.

CONCRETE BRIEFS

Notes from Field and Office

Calibration and Use of Impact Test Hammer.....161
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Calibration and Use of Impact Test Hammer*

By N. G. ZOLDNERS†

The most recent developments in materials testing technique are the nondestructive methods. Considerable attention has recently been focused on the Schmidt concrete test hammer, which is based on an empirical relationship between the strength of concrete and the rebound of a steel plunger which is impinged against the face of the concrete under controlled conditions. The test hammer, its calibration, and use have been described in an earlier JOURNAL paper.‡ This material is offered as addenda to the earlier report.

The calibration chart furnished with each hammer can be used only when material and testing conditions are similar to those in effect when calibration of the instrument was carried out. Each test hammer varies slightly in performance and has to be calibrated for use on concrete made with aggregates produced from a specific source (Fig. 1).

Calibration

The simplest way to calibrate the test hammer is to obtain a sufficiently representative number of impact-rebound readings on standard 6 x 12-in. concrete test cylinders immediately prior to their being broken in a compression testing machine.

As the densest and strongest part of the concrete test cylinder usually is the lower part of the specimen, the bottom face should not be used for calibration purposes. Concentration of coarse aggregates at the bottom will result in erroneously high impact-rebound readings. It is also improper to use the upper part of the test cylinder because of effects of water and air gain at the top surface.

A method of calibration has been worked out which utilizes impact-rebound readings on the cylinder side surfaces.

As a rule only 28-day-old, 6 x 12-in. concrete test cylinders in saturated-surface-dry (SSD) condition are used. After capping, the cylinder is placed in the compression testing machine and an initial load of approximately 150 psi is applied for restraint of the specimen. Impact-rebound readings are made in three vertical lines, 120 deg apart, against the side surface in the middle two-thirds of each cylinder. Obviously erroneous readings due to hitting stone particles or air voids behind apparently sound surface areas are not taken into account. Out of 15 reliable readings the three lowest and three highest are discarded and the average of the nine numbers is the value of the rebound readings for this cylinder, the strength of which is

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*Adapted from a report presented at the ACI 9th regional meeting, Montreal, P. Q., Canada, Oct. 25, 1956.

†Member American Concrete Institute, Senior Concrete Engineer, Mount Royal Paving & Supplies, Ltd., Montreal, P. Q., Canada.

‡Greene, Gordon W., "Test Hammer Provides New Method of Evaluating Hardened Concrete," ACI JOURNAL, Nov. 1954, *Proc.* V. 51, pp. 249-256; discussion in ACI JOURNAL, Part 2, Dec. 1955, *Proc.* V. 51, pp. 256-1 to 256-20.

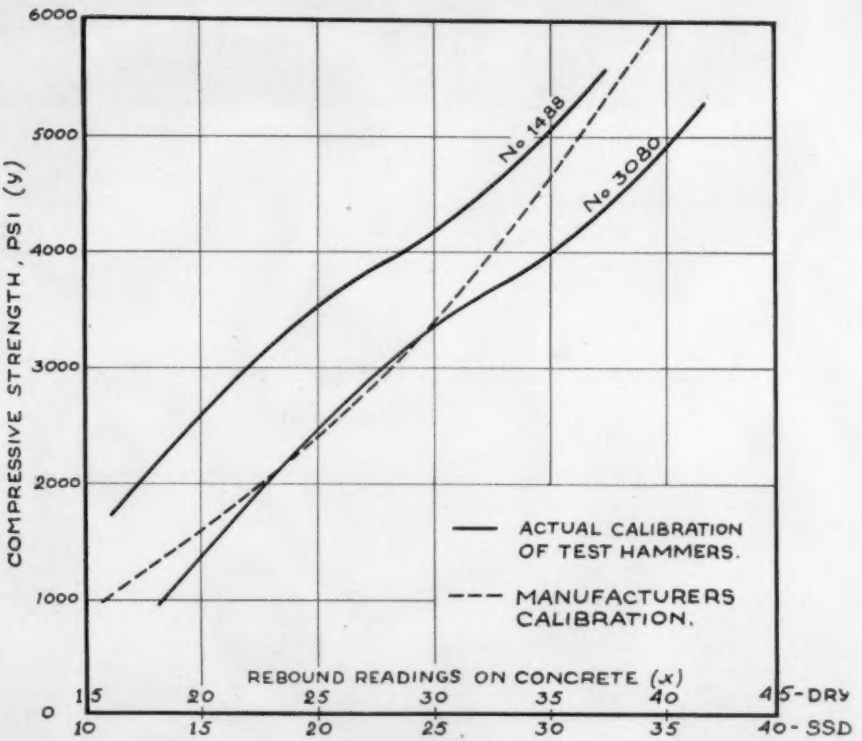

Fig. 1—Manufacturer's calibration compared with actual calibration curves for Schmidt test hammers No. 1488 and 3080

found by testing the specimen to failure. This value of the rebound readings is plotted as in Fig. 2 against the tested cylinder strength. (Readings on curved surface of cylinder and on a surface ground to a plane were compared. No measurable difference was noted.)

After sufficient rebound readings in all concrete strength ranges are obtained and plotted on the graph (Fig. 2), a median curve is drawn, to fit smoothly to the mean values of each group of dots marked (with a small circle in Fig. 2). This mean value is an average of all strength figures showing the same rebound number. Table 1 shows the standard deviation of compressive strength values within each group of tests having the same impact-rebound number. The coefficient of variation is within the accuracy limits of concrete strength fluctuations normally obtained on well controlled jobs and tested as standard test cylinders.

The law of normal probability indicates that in these circumstances we can expect three tests out of 100 to be more than 15 percent below the median curve and an equal number above.*

Fig. 2 shows, if testing is carried out under the same conditions, that impact test hammer rebound readings determine the strength of concrete within a practical limit of ± 15 percent.

These established limits of the probable strength of concrete are good only for a particular set of conditions. In testing con-

*"Recommended Practice for Evaluation of Compression Test Results of Field Concrete (ACI 214-57)", reported by ACI Committee 214, ACI JOURNAL, July 1957, *Proc.* V. 54, pp. 1-20.

TABLE 1—DEVIATION OF COMPRESSIVE STRENGTH VALUES AT SAME REBOUND READINGS, TEST HAMMER NO. 3080

Rebound reading	21	22	23	24	25	26	27	28	29	30	31	32	33	Average
Number of tests	11	6	9	19	46	54	75	59	70	51	28	17	10	37
Standard deviation	221	251	178	246	299	305	260	290	341	325	311	311	242	275
Coefficient of variation	8.5	8.0	5.6	7.7	9.0	8.9	7.0	7.6	8.5	8.0	7.2	7.0	5.4	7.6

crete under field conditions, correction factors have to be used to adjust the test hammer readings to the standard calibration chart, derived from tests on concrete cured and tested under standard conditions.

Use

The main advantage of the Schmidt test hammer is its handy use under field conditions. However, a reliable interpretation of impact-rebound readings demands competent determination of factors to be taken into account in the field testing of concrete structures.

The following precautions should be observed when using the test hammer.

Testing Surface

It is recommended that the test hammer be used only on surfaces, preferably vertical, where concrete was cast against a form. When the formed surface is rough, more

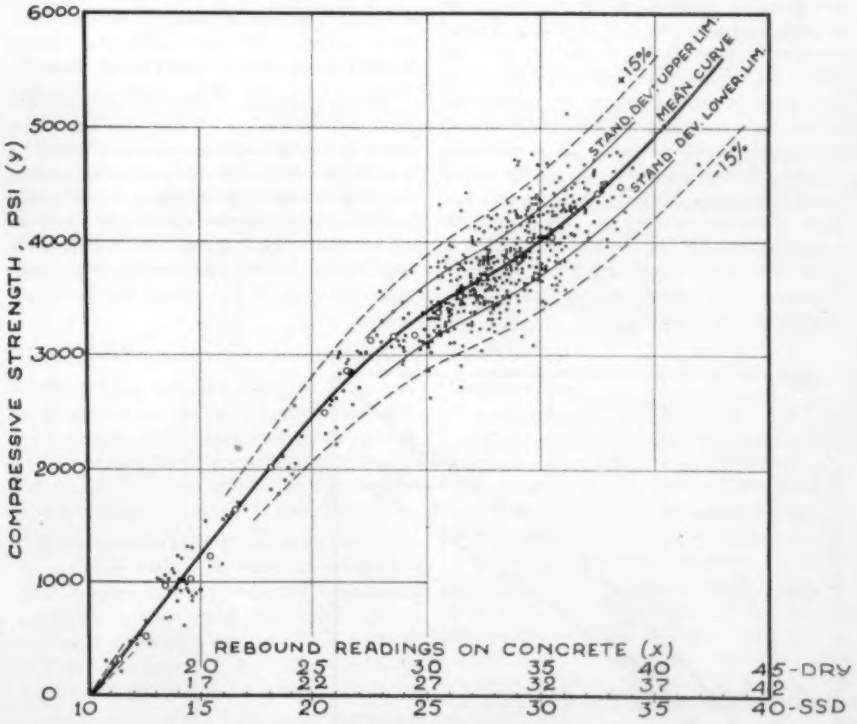

Fig. 2—Calibration chart for concrete made with crushed limestone and natural sand aggregates. Five hundred standard 6 x 12-in. cylinders tested SSD at 28 days. Test hammer No. 3080 was calibrated in horizontal position. Add 5 points for downward direction; deduct 5 points for upward direction

accurate results can be obtained by grinding to a smooth surface with a carborundum stone. Rebound readings on finished floors or rough floated surfaces will not give a true picture of the quality of concrete in the structure itself. Tests made on the underside of a slab are likely to be erroneously high. Test results from the slab top surface should be interpreted with caution. Where the surface has been screeded or wood-floated only, the top layer, as a result of water or air gain, may be appreciably weaker than the underlying concrete. On the other hand, power machine finishing may densify concrete at the surface and lead to erroneously high conclusions as to slab strength.

Impact-Rebound Readings

The instrument has to be held firmly and the plunger pressed exactly perpendicular against the testing surface, applying gradual increase in pressure until the hammer impacts. Obviously erroneous readings, due to hitting stone, air pockets, or honeycomb behind an apparently sound surface should not be taken into account. It is advisable to take 15 readings from one locality within a radius of 6 in. After the three highest and three lowest numbers are discarded, the remainder should not deviate more than 2 points from the average, which will represent the mean value of rebound readings for the concrete at this location.

Size of Specimen

If the concrete section is small, such as a thin wall, slab, or beam, the elastic deformation of the mass under applied impact will lower the rebound readings appreciably. In such cases the member has to be fixed rigidly or backed up by a heavy mass on the opposite side; even then readings may be low.

The hammer is particularly useful in determining strength and uniformity of precast units. It can be used to advantage with pretensioned units in determining when the concrete has attained sufficient strength to permit prestressing. In such tests impact readings should be made only at points of greatest section with the hammer in a horizontal position. Preferred locations are along the sides of top and bottom flanges, end face of beam or girder, or the supports for transverse joists.

Impact Direction of the Test Hammer

The test hammer is normally calibrated in a horizontal direction. When testing downward or upward the rebound is affected by the acceleration due to gravitation forces. Tests show that 5 points have to be added to the reading in the downward direction, and 5 points have to be deducted from the readings in the upward direction to translate these readings to the values for horizontal testing.

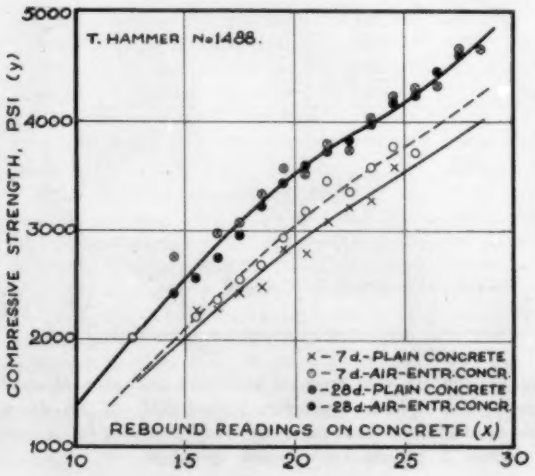

Fig. 3—Calibration curves for plain and air entrained concrete at 7- and 28-days; test hammer No. 1488

Effect of Air Entrainment

Impact-rebound readings were made on 28-day cylinders molded from plain and air-entrained concrete. When the mean values of the several rebound reading ranges were plotted, no significant difference could be found between plain and air-entrained concrete.

Age of Concrete Specimen

Rebound readings were compared on standard test cylinders tested in the saturated-surface-dry condition at two ages (Fig. 3). For equal strength, higher rebound values were found on 7-day-old cylinders as compared to 28-day cylinders. For air-entrained concrete, rebound values for 7-day tests were about 2 to 3 points higher than 28-day tests. For plain concrete the difference was about 3 to 5 points. Up to the age of 7 days, the outside surface of concrete cylinders seems to harden at a faster rate than the internal strength increase. This may be one of the reasons for high rebound readings at 7 days.

Concrete at later ages was not investigated. However, in the testing of old concrete, it would seem desirable to obtain cores from the structure for compressive strength tests and to develop a direct relation between impact test values and actual compressive strengths.

Moisture Condition of Concrete Surface

The varying moisture condition of concrete in structures is the main factor responsible for inconsistency of impact-rebound readings, and for controversies about the evaluation of the impact hammer test results for strength determination of concrete in the field.

It was found that well cured air-dry specimens, when soaked in water and tested in SSD condition, show rebound readings 5 points lower than when tested dry. When the same specimens were left in a room at 70 F and air-dried, they recovered 3 points in 3 days, and 5 points in 7 days.

Although there was a difference of 5 points in rebound readings between SSD and dry concrete at 28 days, the difference was less for concrete at early ages and practically nil for 1 day old concrete.

No long-range tests were conducted, but as reported previously,* differences up to 10 to 12 points in impact-rebound readings have been found in the case of 3-year-old concrete between specimens stored outdoors in wet condition and laboratory dry samples.

Our tests clearly show the need for standardization of concrete testing procedure with respect to specimen moisture content. The degree of saturation of concrete and the presence of surface moisture has a decisive effect on the evaluation of test hammer results.

A practical solution for correcting rebound readings on concrete with different moisture content is the use of three sets of rebound readings—for dry, medium dry, and saturated-surface-dry concrete of the same compressive strength (see Fig. 2). Where the actual moisture condition of the concrete in the field is indeterminate, it might be advantageous to pre-saturate the surface for several hours prior to testing and correlate the rebound readings with the strength for SSD condition.

Coming

in an early "Concrete Briefs" section: "Ultimate Flexural Strength of Prestressed Concrete Beams"

Conclusions

A competent understanding of impact-rebound readings requires that skilled interpretation of all the various factors be taken into account.

The Schmidt test hammer provides a simple, inexpensive method of non-destructive testing of concrete in place. It cannot substitute for standard methods of compression tests, but its accuracy is sufficient to determine probable strength limits of the concrete in the structure and detect low-strength batches, provided it is calibrated properly and used competently by a skilled operator.

*Discussion by Paul Klieger of "Relation of Rebound-Hammer Test Results to Sonic Modulus and Compressive Strength Data," by Perry H. Peterson and Ulrich W. Stoll, *Proceedings*, Highway Research Board, V. 34, 1955, pp. 387-399.

Concrete's Etymological Offspring*

By NOCH EIN LESER

Eighteen additions to the "concrete word family" are presented below, along with their meanings and sources. Previously published lists have mentioned 346 other such words.

Word	Where used
Consteel	(Steel joists) Concrete Steel Co., New York, N. Y.; Steel Joist Institute catalog (1952) p. 18
Diamond-Crete	(Hardener) Concrete Service Co., Philadelphia, Pa.; *Sweet's Architectural File*, 1956, Sec. 9/Co
Dryerete	(Admixtures to increase density and water-repellency) C. G. Pardee Co., Inc., Long Island City, N. Y.; *Sweet's Industrial Construction File*, 1956, Sec. 5/Pa
Duffcon	(Precast concrete slabs and planks) Duffy Concrete Products, Inc., Tenafly, N. J.; *Sweet's Architectural File*, 1956, Sec. 2e/Duf and 2e/Du
Emeri-Topcrete	(Aggregate for surface-hard concrete) Walter Maguire Co., New York, N. Y.; *Contractors and Engineers*, Aug. 1956, p. 110
Flex-Krete	(Resurfacer and treatment) Concrete Service Co., Philadelphia, Pa.; *Sweet's Architectural File*, 1956, Sec. 9/Co
Kompocrete	(Mastic floor) Kompolite Co., Inc., Long Island City, N. Y.; *Sweet's Industrial Construction File*, 1956, Sec. 3h/ke
Magnecon	(Fire brick for rotary cement kilns) Harbison-Walker Refractories Co., Pittsburgh, Pa.; *Sweet's Industrial Construction File*, 1956, Sec. 6b/Ha
Magnecon (continued)	Consett Iron Co., Ltd., Consett, County Durham, England; *The Engineer*, June 1, 1956, advertisement, p. 39
Mixcrete	(Integral hardener) C. G. Pardee Co., Inc., Long Island City, N. Y.; *Sweet's Industrial Construction File*, 1956, Sec. 5/Pa
Nocrete	(Fiber conduit designed for burial without concrete encasement) Orangeburg Mfg. Co., Inc., Orangeburg, N. Y.; *Sweet's Industrial Construction File*, 1956, Sec. 11c/Or
Parcrete	(Asphalt emulsion resurfacer) C. G. Pardee Co., Inc., Long Island City, N. Y.; *Sweet's Industrial Construction File*, 1956, Sec. 5/Pa
Platikrete	(Local advertiser's version of Plasticrete); the *Hartford* (Conn.) *Courant*, June 30, 1956, p. 5
Resista-Crete	(Paint) Waterlox Div., Empire Varnish Co., Cleveland, Ohio; *Sweet's Architectural File*, 1956, Sec. 15/Em
Steel-ceil-crete	(Deck floors for libraries) Art Metal Construction Co., Jamestown, N. Y.; *Sweet's Architectural File*, 1956, Sec. 23f/Ar
Sulfacrete	(Sulfate-resisting cement) Cement Marketing Co., Ltd., London; *Concrete and Constructional Engineering*, July 1956, p. lxxxiv
Supercrete	(Sealer) C. G. Pardee Co., Inc., Long Island City, N. Y.; *Sweet's Industrial Construction File*, 1956, Sec. 5/Pa

*Previous lists appeared in ACI JOURNAL, *Proc.* V. 47, Nov. 1950, p. 263, May 1951, p. 752; *Proc.* V. 49, Mar. 1953, pp. 682-684; *Proc.* V. 50, June 1954, pp. 894-897; *Proc.* V. 51, Apr. 1955, pp. 813-817; *Proc.* V. 53, Aug. 1956, pp. 226-228.

Temp'O'Crete (Insulating concrete) Temp-'O'Crete International Corp., Brooklyn, N. Y.; *Thomas' Register*, 1956

Vitricon (Glazing spray for concrete block) Vitricon, Inc., Long Island City, N. Y.; *Engineering News - Record*, Aug. 16, 1956, p. 64

Problems and Practices

A series relating to "down-to-earth, everyday" concrete problems which attempts to give brief answers to the more common (and sometimes uncommon, too) questions asked about concrete design and construction practices.

To some, the answers will seem simple and obvious; others may prove to be extremely controversial.

All ACI members are invited to participate—either by submitting an inquiry, or even better, by telling JOURNAL *readers how an intriguing problem was solved. It may well be that readers will be able to suggest more practical solutions than those presented.*

Q. Is membrane curing a satisfactory and adequate procedure for the interiors and exteriors of buildings? What membrane materials may be used?

A. Membrane curing is most commonly applied to floors and paving. It includes impervious coverings as well as liquid sealing compounds. Best practice for exterior use requires a white-pigmented type to reflect the heat from the sun. Liquid membranes cannot be used on rough floor slab base courses nor walls intended to receive a future bonded finish. Tests seem to indicate that any concrete with a water-cement ratio more than 5 gal. per sack contains enough free water to cure itself adequately if evaporation is prevented. On richer concretes additional water should be supplied during the initial curing for optimum results.

Liquid membrane-forming compounds are eminently suited for curing walls in lieu of leaving forms in place. Structurally it is usually safe to remove wall, beam side, and column forms in about 12 hr. Most specifications require such forms to be kept in place 3 to 7 days unless other curing provisions are undertaken. Liquid membrane-forming compounds are usually less expensive than wet burlap, and eliminate need for repeated rewetting and continuing inspection. Where the remaining traces of compound will not be objectionable in appearance or interfere with bond of future finishes, this curing method can lead to real economy, permitting faster reuse of forms.

ACI Committee 617's "Proposed Revision of Specifications for Concrete Pavements and Concrete Bases"* recommends that liquid sealing compounds conform to the requirements of "Specifications for Liquid Membrane-Forming Compounds for Curing Concrete" (ASTM C 309), that waterproof paper and impermeable sheets other than paper conform to the water retention requirements of "Specifications for Waterproof Paper for Curing Concrete" (ASTM C 171).

The Testing and Research Division of the Michigan State Highway Department reports that "Michigan specifications require a minimum total non-volatile content of 60 percent in the white compounds, which includes both pigment and non-volatile material in the vehicle. We have found that this requirement results in the production of material with better consistency or 'body', and provides a factor of safety against possible non-uniformity of application. Our specifications also prohibit the use of waxy compounds, which have been found to adversely affect the drying time and water retention of the film at summer temperatures. These provisions are not found in the ASTM specifications, but we consider them necessary in the procurement of a quality product."

The use of polyethylene sheeting for membrane curing is relatively new. Both clear and

*ACI JOURNAL, Apr. 1957, *Proc.* V. 53, pp. 917-946.

white material is available in various widths. The 1956 specification of the New Jersey State Highway Department for this material is:

Polyethylene sheeting, white—White polyethylene sheeting shall consist of a single sheet manufactured from virgin resin with no scrap or additives. The sheeting shall be white, furnished in rolls, and shall be free of visible defects and shall have a uniform appearance. It shall be sufficiently strong and tough to permit its use under the conditions existing on highways and structural work without tearing or otherwise becoming unfit for use for which it is intended.

Moisture retention—When tested in accordance with the current method of test C 156 of the ASTM there shall not be a moisture loss greater than 0.055 g per sq cm of the mortar specimen surface based on the amount of water in the mortar at the time the curing medium is applied.

Reflectance, apparent daylight—The sheeting shall have a length of not less than 100 ft and not more than 300 ft, a normal thickness of not less than 4 mils, (0.004 in.) and a width of not less than 14 ft unless otherwise specified.

Tensile strength—The sheeting shall conform to the following requirements as to tensile strength at 77 F:

Machine direction
Minimum
1700 lb
Transverse strength
Minimum
1200 lb

Elongation—The sheeting shall conform to the following requirements as to elongation at 77 F:

Machine direction
Minimum
225 percent
Transverse strength
Minimum
350 percent

Certificate of analysis—The manufacturer shall furnish with each shipment a certified analysis, giving the results of tests required by these specifications.

* * * *

Q. Does the use of calcium chloride in concrete have any harmful effect?

A. This question is best answered by a direct quotation from the ACI Standard "Recommended Practice for Winter Concreting": "Use of 1 percent of calcium chloride (by weight of the cement) is recommended in weather cold enough for mean temperatures to be below 40 F, to obtain higher concrete strength at the end of the period of protection." No more than 2 percent of calcium chloride by weight of the cement should be used. Normal protection periods should be doubled to secure best durability when it is preferred not to use calcium chloride.

Calcium chloride should not be added to the mix in flake form. It is convenient to add it dissolved in one part of the mixing water as a solution containing 1 lb per quart. Calcium chloride should not be used when sulfate-resisting concrete is required, when stray electrical currents may cause corrosion, or when stress corrosion may occur. These exceptions to the normal recommended winter practice will most commonly affect sanitary sewage facilities, some foundations and other concrete exposed to sulfate ground water, and prestressed concrete. For these exceptional cases, an additional sack of cement per cu yd is recommended to secure high early strengths.

"The use of salts, chemicals, or other foreign materials in the mix to lower the freezing point of the concrete should not be permitted," to quote further from the winter concreting standards.

Committee 212 in "Admixtures for Concrete" reported as follows (ACI JOURNAL, Oct. 1954, *Proc.* V. 51, pp. 113-148): (1) the shrinkage of concrete is generally increased by calcium chloride; (2) the *rate* of heat evolution is increased and, where temperature differentials within the concrete are important factors, the effect of this property should be considered; (3) the use of accelerators in warm concretes as in hot weather concreting may result in such rapid stiffening as to impede placing and finishing; (4) it increases expansion caused by the alkali-aggregate reaction; (5) the use of calcium chloride in the dry form is not advised . . . may result in . . . pop-outs.

CURRENT REVIEWS

of Significant Contributions in Foreign and Domestic Publications

Bridges

Field tests on a prestressed concrete multi-beam bridge

A. Roesli, A. Smislova, C. E. Ekberg, Jr., and W. J. Eney, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 152-171

Describes field tests consisting of moving loaded trucks on a 32-ft span prestressed multi-beam bridge consisting of rectangular units with two circular cores placed adjacent to each other and provided with shear keys of dry-packed grout in the joints between units and a 1 in. round lateral tie rod at the center of the 32-ft span to transfer shears. Principal conclusion is that the over-all behavior of the bridge approached that of a homogenous plate.

Some methods of extending the life of bridges by major repairs or strengthening

Jack Denis West, *Proceedings*, Institution of Civil Engineers (London), V. 6 (session 1956-57), Feb. 1957, pp. 183-202 (discussion, pp. 203-215)

Reviewed by Aron L. Mirsky

Discusses primarily repairs to abutments, foundations, and spandrel walls of old British railway bridges and archs. Controlling factor is minimum interference to and interruption of railway traffic.

Puddefjord bridge (in Norwegian)

Caspar Trumpy, *Nordisk Betong* (Stockholm), V. 1, No. 1, 1957, pp. 9-35

Reviewed by Margaret Corbin

Puddefjord Bridge has a center reinforced concrete arch span of 150.1 m with a rise of 25.42 m, a ratio of 0.170. It was designed for the highest loading category used in Norway as a fixed-end, hingeless arch. The arch is divided into two sections, each 4.40 m wide and separated 2.10 m, joined at the crown and at the abutments. At the crown the arch is 1.38 m thick; at the base, 2.45 m. Axis of the arch follows the thrust line for dead load, and the moment of inertia of the cross section decreases linearly from abutment to crown, with slight adjustments.

Reinforced with 0.6 percent deformed bars, the arch was cast in elements 9.40 m long, with 1.60 m joints between. Reinforcing was spliced in the joints. Concrete in the arch is batched with 475 kg cement per cu m; concrete with 600 kg cement per cu m was used for the joints. Concrete was moist cured 3 months by means of a water sprinkling arrangement.

Moments in simply supported skew I-beam bridges

T. Y. Chen, C. P. Siess, and N. M. Newmark, *Bulletin* No. 439, University of Illinois Engineering Experiment Station, Jan. 1957, 72 pp., $1

An analytical study of the behavior of the simply supported skew I-beam bridge which consists of a slab of uniform thickness continuous over steel beams which are parallel to the direction of traffic.

The solution of the finite difference equations was performed on an electronic digital computer. Moments in the beams and the slab at various locations were determined for a moving concentrated load placed anywhere on the bridge. Extensive tables giving influence values for these quantities are presented.

Maximum live load midspan beam moments were computed for the standard AASHO truck loadings. Bridges considered have spans varying from 25 to 80 ft. Empiri-

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cal formulas have been developed for estimating maximum live load midspan beam moments in any skew bridge having proportions lying within the ranges considered. Approximate rules for obtaining dead load beam moments in skew bridges are also given.

Full scale tests of concrete bridge rails subjected to automobile impacts

J. L. BEATON, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 251-267 (including discussion)

Describes a series of full scale tests conducted by the California Division of Highways using test cars operated by remote control radio at speeds between 50 and 60 mph at the time of collision. The data recorded include the effect upon the rigid concrete barriers and upon the car, and also high speed moving pictures of the collision itself.

Firth of Forth road crossing

The Engineer (London), V. 203, No. 5273, Feb. 15, 1957, pp. 257-259

Reviewed by ARON L. MIRSKY

Description of two schemes proposed for crossing the Firth of Forth: a suspension bridge, and a concrete underwater "tube" supported above the river bed by piles. Tube would be built of precast sections, floated out into position and sunk onto the piles; due to depth of bed and type of bed materials, usual tunnel in trench cannot be adopted for main portion of crossing. Permanent caisson piles, of "Gambia" type (cf. *Proceedings*, Institution of Civil Engineers (London), Part II, V. 4, No. 1, Feb. 1955, pp. 119-136; "Current Reviews," ACI JOURNAL, Feb. 1956, *Proc.* V. 52, p. 692) but using steel shells rather than reinforced concrete, are constructed after tube is in place.

Suspension bridge scheme has been adopted, for reasons which are given in article and which have been attacked. Article does not discuss these, this being reserved for an editorial on p. 240 of the same issue.

Voulte viaduct on the Rhone River

French Economic and Technical Bulletin, No. 3, 1957, pp. 14-17

The first use of prestressed concrete for a major railroad bridge in France. The design features inverted A-frame abutments, 197-ft span, and straight beams limited to a thickness of 9.4 ft for navigation requirements. The viaduct totals 984 ft in length between banks of the Rhone and required 4000 tons of prestressing for each cross beam.

Field measurements on a composite slab girder bridge

G. HONDROS and G. MARSH, *Civil Engineering and Public Works Review* (London), V. 52, No. 611, May 1957, pp. 557-559

Presents the results of field measurements on a skew bridge composed of steel girders composite with a reinforced concrete deck slab, anchorage of the two being achieved by using steel strips welded to the top flanges of the girders to resist the composite shear. A description of the design and details of the construction are given, as well as results of field test under load.

Construction

Use of Gunite as a structural material: A survey of developments, 1930-1955

T. WHITLEY MORAN, *Civil Engineering and Public Works Review* (London), V. 51, No. 600, June 1956, p. 674

HIGHWAY RESEARCH ABSTRACTS Nov. 1956

Some of the main categories of work for which Gunite is commonly used include water-retaining structures, steelwork protection, tank linings, prestressed concrete tanks, and shell roofs. The greater part of the paper is, however, devoted to two main developments of structural interest. These are the comprehensive reconditioning of reinforced concrete structures, and the strengthening of weakened structures. Each of these aspects is discussed with the aid of a number of practical examples.

The building frame and the steel shortage

A. L. L. Baker, *The Engineer* (London), V. 203, No. 5274, Feb. 22, 1957, pp. 286-288
Reviewed by Aron L. Mirsky

Author argues that Great Britain should follow the lead of the rest of the world and adopt reinforced concrete frames for tall buildings, reserving structural steel for buildings over, say, 30 stories. Points of superiority of concrete are presented, and objections sometimes raised are refuted. Author points out that if more concrete is to be used, provision must be made for increased production of reinforcing bars.

Dams

Allt-na-Lairige prestressed concrete dam

James A. Banks, *Proceedings*, Institution of Civil Engineers (London), V. 6 (Session 1956-57), Mar. 1957, pp. 409-428 (discussion, pp. 428-444)
Reviewed by Aron L. Mirsky

Dam, on one of the smaller projects of the North of Scotland Hydro-Electric Board, has a maximum height of 73 ft and is 1360 ft long; major portion of length is prestressed, only spillway and extreme end blocks being of gravity section. Prestressing was selected on basis of cost estimates, which showed a cost saving over a gravity dam, permitting an increase in the economic height and hence storage capacity. Provision was made in design for long-term observation of behavior of bars and of dam proper.

Design for a narrow valley: Salime hydro-electric station is built beneath the spillway

Engineering (London), V. 183, No. 4750, March 22, 1957, pp. 354-356
Reviewed by Aron L. Mirsky

Brief description of an interesting solution to the problem of a dam and power plant in a very narrow, deep valley subject to landslides and avalanches. Dam is of arch gravity type, with a uniform triangular section throughout and a crest length of 825 ft and height of 434 ft above foundation. Power plant, of the "one floor type," is situated below spillway (which forms its roof), and of necessity has a curved longitudinal axis to conform to curvature of dam. The four turbo-alternator sets lie on this curve, and are served by an overhead crane which also moves along an arc.

Hydroelectric power plant with reservoir at Jablanica—first concrete arch dam on Neretva River (in Serbian)

Gradan Caric, *Naše Gradevinarstvo* (Belgrade), V. 11, No. 1, Jan. 1957, pp. 2-14
Reviewed by J. J. Polivka

Detailed description of the plant designed under the direction of Jaroslav Cerni. Arch dam, 250 ft high, was selected after study of two alternates (gravity dams in concrete and stone). The dam has some characteristics of a typical constant-angle dam, system Jorgensen, and carries a highway bridge in reinforced concrete. Water from the lake, about 6 miles long, is conveyed by two parallel pressure tunnels to the surge tower and from there to the underground-hall with six Francis turbines, 31,000 hp per unit, generators and transformers.

Concrete dam was cast in 14 blocks with vertical joints; 70 percent portland cement was mixed with 30 percent slag cement, total 430 lb per cu yd. Aggregate maximum size was 3 in. Water-cement ratio of 0.56 was used. Tunnels, from 14 to 20 ft in diameter, were built in concrete (maximum wall thickness 32 in.), partly in reinforced concrete (14 in. wall thickness, with 2¾ in. Torkret waterproofing).

Design

Ultimate load theory applied to the design of reinforced and prestressed concrete frames

A. L. L. Baker, Concrete Publications, Ltd., London, 1956, 92 pp., 18s
The Structural Engineer Jan. 1957

Deals with reinforced and prestressed concrete beams and frames, giving the basic principles of ultimate load theory and design.

The all important characteristics of the method is that it is based on the load producing failure or collapse of the whole structure. The mode of failure is discussed together with a choice of a suitable load factor or factor of safety. The book contains many examples of the conception of plastic hinges, the introduction of which enables use to be made of the maximum strength and econo-

mies of structures designed by the ultimate load method.

Anisotropic cylindrical shells

A. CHRONOWICZ, *Civil Engineering and Public Works Review* (London), V. 52, No. 611, May 1957, pp. 546-549

Develops the theory for analysis of cylindrical shells anisotropic by virtue of stiffening ribs, either in the longitudinal or circumferential direction. These ribs may prove desirable in designs generally, to eliminate the possibility of buckling, to reduce excessive deflections in the case of large spans, or to accommodate a large number of perforations. The theoretical analysis is illustrated by an application, the design of a two-span structure for an English brewery.

On the internal pressure in prestressing with bond and in concrete prestressed with steel strands (Über die innere Anpressung bei Vorspannung mit Verbund und bei Stahlsaitenbeton)

W. SWIDA, *Der Bauingenieur* (Berlin), V. 31, No. 2, Feb. 1956, pp. 52-55

Reviewed by ARON L. MIRSKY

Develops expressions for stresses in concrete normal to bonded prestressing elements. A numerical example indicates these stresses to be high, but author argues that no structural harm can occur in general. Reasons for this conclusion are given.

Stability of concrete structures submitted to long-time loads (in Swedish)

LARS OSTLUND, *Nordisk Betong* (Stockholm), V. 1, No. 1, 1957, pp. 77-87

Reviewed by MARGARET CORBIN

This paper deals with two problems relating to the design of concrete structures subjected to long-time loads: the problem of stability, and the problem of calculating the magnitude of the coefficient of increase in moment. The treatment of these problems is based on several assumed mathematical expressions which represent the creep of the concrete.

Results indicate that the problems in question can be dealt with by means of methods analogous to those used in the case of short-time loads, with the only exception that an idealized modulus of elasticity is introduced instead of the actual modulus.

Calculation of an oblique slab by the method of finite differences (in French)

R. DEKEYSER and J. CAPEL, *Annales des Travaux Publics de Belgique* (Brussels), No. 4, 1955, pp. 5-40

APPLIED MECHANICS REVIEWS May 1957 (Seydel)

The deck plate of an oblique prestressed concrete bridge (combined highway-railroad bridge) is stiffened by longitudinal and transverse girders.

Numerical results (deflections and bending moments) are represented by diagrams and compared with those of a rectangular plate; diagrams show a considerable increase of clamping moments in the obtuse angles of the parallelogram plate.

Design of earthquake-resistant buildings—1 and 2

The Indian Concrete Journal (Bombay), V. 30, No. 10, Oct. 1956, pp. 315-321; No. 12, Dec. 1956, pp. 398-404

The first article of series reviews fundamental knowledge on earthquake action and fundamental principles of aseismic construction. The second paper deals exclusively with design and details for earthquake resistance in reinforced concrete buildings.

Concrete slabs reinforced for torsion (in Danish)

JORGEN NIELSEN, *Nordisk Betong* (Stockholm), V. 1, No. 1, 1957, pp. 57-76

Reviewed by MARGARET CORBIN

The unit torsional moment m_t cannot be determined directly by distributing a total torsional moment over the length of section due to the fact that the vertical shearing forces also contribute to the total torsional moment. The vertical shearing forces in the interior of the section are determined by an equation; the concentrated forces produced along free and simply supported slab edges are determined by a second equation. Figures illustrate lattice systems to take up simple torsion in rectangular slabs reinforced by double nets in top and bottom, the reinforcement placed at 45-deg angles to main moments.

In the succeeding section, the safety against failure is investigated for a given reinforced slab exposed to simple torsion. Finally the approximation is discussed which is made by using unequal reinforcement in the two directions of the slab for the torsional moment and it is shown that this produces additional stresses in the reinforcement.

Design of statically indeterminate structures by the "plastic-hinge" method

R. GARTNER, *Concrete and Constructional Engineering* (London), V. 52, No. 5, May 1957, pp. 161-174

Illustrates theory of limit analysis applied to structural steel and reinforced concrete. In particular investigates the behavior of steel and reinforced concrete hinges as plastic materials.

Simplified design method for reinforced concrete pipe under earth fills

D. P. BABCOCK, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 210-237 (including discussion)

Proposed criteria for the design and installation of reinforced concrete pipe culverts are being studied by the U. S. Bureau of Public Roads in cooperation with Prof. M. G. Spangler of Iowa State College. In connection therewith, an attempt has been made to simplify the prevailing methods of computing the necessary pipe lengths. This paper explains the method. The proposed criteria are appended together with some suggested tests.

Materials

Epoxy adhesives as structural repair material

HERBERT A. ROONEY and E. D. BOTTS, *California Highways and Public Works*, V. 35, No. 9-10, Sept.-Oct. 1956, pp. 48-50

HIGHWAY RESEARCH ABSTRACTS Mar. 1957

Field and laboratory research projects were initiated to ascertain if epoxy-thiokol adhesives could be used for miscellaneous repairs on concrete bridges and highway surfaces.

Several 6 x 6 x 30-in. concrete beams were broken in flexure using third-point loading. After breaking, the two pieces of each beam were cemented together with an epoxy-thiokol adhesive and allowed to cure for 7 days at temperatures not less than 70 F. The cemented beams were again subjected to flexure, the breaks occurring in sections of the beams other than the repair point.

Several 6 x 6 x 30-in. concrete beams were broken in flexure, one end of each broken beam section was coated with the adhesive, and fresh concrete immediately cast against it. The reconstituted beams were approximately one-half old and one-half new concrete made to the original dimensions. When the beams were again subjected to test all breaks occurred in the concrete, the adhesive joint between the old and new concrete sections remaining intact.

In May 1955 a section of abraded concrete highway surface was repaired using the adhesive. As of July 1956 all repaired spots were still in excellent condition.

Laboratory tests showed that the epoxy resin adhesive had excellent properties for the grouting of steel bolts in concrete. Bolts of $\frac{5}{8}$-in. diameter were grouted into concrete monuments in 2-in. diameter holes 8 in. deep. After proper curing of the adhesive, attempts were made to pull out the bolts by application of a tension load. In all cases the bolts broke in tension at 14,000-lb load, the grout remaining intact.

Freeze-thaw durability of Michigan concrete coarse aggregates

F. E. LEGG, JR., *Bulletin* No. 143, Highway Research Board, Mar. 1957, pp. 1-13

Presents laboratory freeze-thaw results as well as field observations on the major Michigan aggregate sources and is considered a first step in formulation of a policy regarding acceptance of future aggregates of unknown service behavior.

Laboratory evaluation of coarse aggregate durability is achieved by following a rigidly standardized procedure wherein the aggregates are graded and placed in air-entrained concrete in a vacuum saturated condition. The ASTM rapid cycle of freezing in air to 0 F and thawing in 40 F water is used. Failure is considered to have occurred when the sonic modulus of 3 x 4 x 16-in. specimens has decreased 30 percent. Durability factors at 300 cycles of freezing and thawing appear to adequately describe the behavior of the aggregates.

Effect of heavy-media separation on durability of concrete made with Indiana gravels

RICHARD D. WALKER and J. F. MCLAUGHLIN, *Bulletin* No. 143, Highway Research Board, Mar. 1957, pp. 14-26

Indiana's present field practice is to blend crushed stone (for the larger sizes of the coarse aggregate) with gravel. Also, the development of commercial heavy-media sep-

aration methods makes worthy of consideration the more extensive use of these methods for the improvement of gravel aggregates. These two factors form the basis of this laboratory research.

Polyvinyl acetate emulsion as an addition to cement

Cement and Lime Manufacture (London), V. 28, No. 4, 1955, pp. 50-51; *Journal of Applied Chemistry*, V. 6, Part 6, June 1956, p. i-786
HIGHWAY RESEARCH ABSTRACTS
Feb. 1957

The effects on the properties of cement mortars of adding 16 to 20 percent of polyvinyl acetate (p.v.a.) emulsion to portland cement is examined. The results (weight-percent) show that maximum improvement in properties is obtained when the mortars are cured in air at ordinary temperatures and humidities. There are thus obtained, particularly when the p.v.a.–cement ratio is 0.2, improved workability, tensile and bond strengths, extensibility, and resistance to impact, abrasion and corrosion. It is suggested that these mortars could be used for floor toppings and road surfaces, high-strength wall and ceiling plasters, masonry surfaces where self curing and high bond strengths are required, and certain tanks and pipes where tensile strength is important.

Early high-strength cement (in German)

I. I. CHOLIN, *Silikattechnik* (Berlin), V. 7, No. 10, 1956, pp. 431-435
CERAMIC ABSTRACTS
May 1957 (Hartenheim)

Theoretical investigations and practical experiences in the USSR with cements having high strength after only 1 to 3 days are reported. Russian standards and testing methods for such cements are compared with those of other countries. In production, an optimum mineralogical composition should be the objective, in which $C_3S + C_3A$ should not be less than 60 percent and the C_3S content should be about 55 percent. Iron portland cement based on alumina-rich or acid slags should be ground to a fineness of at least 5000 sq cm per g. The amount of gypsum must be adapted to the fineness and setting rate of the aluminates. Addition of calcium chloride or heat treatment increases strength. The results are detailed in tables and curves.

Behavior of low-alumina (sulfate-resistant) portland cements in sulfate solutions (in Russian)

V. V. KIND, *Trudy Soveshchaniya Tsement. i Beton. Gidrotekh. Stroitel.* (Leniadat), 1953, pp. 35-52; *Chemical Abstracts*, V. 49, No. 5, 1955, 3502f
CERAMIC ABSTRACTS
Mar. 1957

Low Al_2O_3 portland cements satisfying the specifications for sulfate-resistant portland cements are appreciably more resistant to sulfate solutions than ordinary cement, but they are noticeably inferior in this respect to certain pozzolan cements and to slag portland cement made with basic blast-furnace slag.

Hydraulic separators for grading sands and powders

The Engineer (London), V. 203, No. 5271, Feb. 1, 1957, pp. 187-188
Reviewed by ARON L. MIRSKY

Description of commercial plant developed by Theodore Eder of Austria and used successfully in the aggregate preparation plant of the Glockner-Kaprun hydroelectric scheme (see *The Engineer*, Sept. 21, 1956, pp. 417-419; "Current Reviews," ACI JOURNAL, Apr. 1957, *Proc.* V. 53, p. 1015). Apparatus, based on principle of Crooke's elutriator, is designed to separate the fractions below about ⅛ in. maximum size; units can be variously combined to suit requirements of any particular job.

Sodium benzoate in concrete

J. I. M. LEWIS, C. E. MASON, and D. BRERETON, *Civil Engineering and Public Works Review* (London), V. 51, No. 602, Aug. 1956, pp. 881-882
HIGHWAY RESEARCH ABSTRACTS
Apr. 1957

A process for the use of sodium benzoate as a corrosion inhibitor to protect the steel in reinforced concrete under certain conditions has been patented by the North Thames Gas Board. A suitable technique for the estimation of sodium benzoate in concrete has been developed, and the method applied to samples taken from a reinforced concrete structure that has been standing for 5 years in a corrosive atmosphere.

Sodium benzoate has little effect on the physical properties of concrete, except its compressive strength, and this factor could be counteracted by modifying the mix proportions to suit specified requirements. It was established that sodium benzoate re-

mains as such in concrete. Further work is in progress to determine its inhibitive power and to compare it with other inhibitors.

Recent technical developments in processing slag products

B. M. Pearson, *Rock Products*, V. 59: No. 2, Feb. 1956, pp. 112-116, 121, 145, 148-154; No. 3, Mar. 1956, pp. 78, 80, 98

Ceramic Abstracts
July & Aug. 1956 (Barbour)

Surveys the various processes for the granulation of blast-furnace slag and considers the characteristics of the products obtained. The hydraulic properties are measured in the percentage content of the devitrified fraction of the slag and the moisture content. The moisture content is important and should be kept as low as possible. The temperature of the water is also important in the granulation process.

Gives figures on the German use of blast-furnace slags. Data are given on the properties and costs of cement made from various combinations of slag and portland cement clinker. Difference in millability of various slags must be considered in studying cost data. (Part 3 of this series was reviewed in ACI Journal, Apr. 1957, *Proc.* V. 53, p. 1020.)

Work-hardened twisted reinforcing steel—Part 2

Kurt Billig, *Civil Engineering and Public Works Review* (London), V. 52, No. 608, Feb. 1957, pp. 173-175

Discusses the bond strength and concrete cracking which accompanies various working stresses permissible under European codes in work-hardened twisted reinforcing steel, and the value of end hooks and bends. It is stated that the cold twisted bars may be effectively welded to develop full strength, but that such welding requires great care, and so the general recommendation is that the design stresses in welded joints of work-hardened steel should not exceed the stresses permissible for ordinary mild-steel bars. Fatigue strength is said to be improved proportionately as well as static strength. The conclusion is that European experience in the past 10 years has demonstrated conclusively that high-strength concrete and high-strength reinforcement may both be used in combination at working stresses to take advantage of these high strengths. A bibliography of 12 references is included. (Part 1 was reviewed in ACI Journal, May 1957, *Proc.* V. 53, p. 1094.)

Chemical disintegration of cement products (in Afrikaans)

J. H. P. van Aardt, *South African Industrial Chemist*, V. 10, 1956, pp. 211-220

Ceramic Abstracts
Apr. 1957 (Frankel)

Reviews the chemical and mineralogical composition of three types of hydraulic cements, the hydration of cements and cement minerals, and the chemical processes which take place between cement products and aggressive materials. Reactions of carbonated waters, sulfates, chlorides, and ammonium salts on cements are given. Brief mention is made of the development of new types of cements, methods for reducing the vulnerability of portland cements, and surface treatment to reduce disintegration.

Protective coatings for atmospheric use: Their surface preparation and application requirements, physical characteristics and resistances

NACE Technical Unit Committee T-6B, *Corrosion*, V. 13, No. 3, Mar. 1957, pp. 85-93

Highway Research Abstracts
May 1957

Detailed information is given on the application requirements, physical characteristics, and resistances of 22 different types of binders. Materials discussed include drying oil paints, ester gum-oil, straight phenolic oil varnish, modified phenolic oil varnish, straight alkyd varnish, modified alkyd varnish, expoxies, chlorinated rubber, vinyls, metallic silicate (zinc loaded), six coal tar coatings, and five asphalt coatings. Information pertaining to the following points is given for each material: recommended surface preparation, number of coats, approximate dried film thickness of each coat, approximate air-drying time before recoating, usual method of application, application surface temperature, thinners, general flammability classifications, compatability with other types, abrasion resistance, gloss retention, resistance to petroleum oil and grease, maximum temperature limitations, approximate storage life under normal conditions, and health hazards.

Pavements

Current practice questionnaire: Jointing, fillers, sealers, and equipment

A. A. Anderson, *Bulletin* No. 138, Highway Research Board, Feb. 1957, pp. 11-24

Summarizes the replies received from 43 states, District of Columbia, Hawaii, Army, Navy, and CAA. Highway engineers and producers of highway materials should find it worthwhile to study this tabulation.

Resurfacing and patching concrete pavements with bonded concrete

E. J. Felt, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 444-469

Describes laboratory bond tests, experimental field projects, a survey of projects in use, and recommends best practice based upon conclusions drawn from these tests and experience. Shear tests indicate that bond strengths of 400 psi or more may be developed but that strengths as low as 200 psi are adequate. Best bond was obtained when the base concrete was dry and grouted. Satisfactory service records of bonded resurfacing as thin as 1 in. date back to 1913.

Investigation of load transfer characteristics of dowels

J. R. Keeton, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 147-151

A study on dowels used in airfield pavement expansion joints. Field installation of instrumentation is discussed. Single wheel loads of 50,000 lb for the preliminary test were applied by a specially designed cart.

Pennsylvania builds its first continuously reinforced concrete pavement

Roads and Streets, V. 100, No. 1, Jan. 1957, p. 76

A two-mile segment of dual-lane highway, continuously reinforced, has been completed in Pennsylvania. It is hoped that the additional reinforcement required will result in reduced cost by elimination of the standard highway joints, their sealing and maintenance cost. It was also stated that standard thickness of concrete in present day heavy traffic highways is 10 in. By using twice as much reinforcing steel in the form of mats, it is expected that the thickness of the slab can safely be cut to 7 in., resulting in an over-all lower initial cost.

Cleaning and resealing of concrete pavement joints by contract

Fred E. Sternberg and Warren M. Creamer, *Bulletin* No. 138, Highway Research Board, Feb. 1957, pp. 1-10

Describes a program established by the state highway department of Connecticut for cleaning and resealing joints in concrete pavement. In addition to resealing joints a 2-in. relief joint was cut in the pavement every 1000 ft to provide expansion space to avoid hot weather blow-ups.

Model study of rigid pavement behavior under corner and edge loadings

P. F. Carlton and R. M. Behrmann, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 139-146

Presents the results of laboratory tests on small scale models compared with theoretical analyses of critical stresses resulting from single wheel loadings at free corners and free edges of rigid pavement slabs. The testing involved measurement of strains in the plaster model slab under static loading. A close agreement was obtained with the behavior predicted by Westergaard's theoretical analysis. Critical stresses resulted from the edge loading, maximum corner stresses being only 45 to 55 percent of maximum edge stresses.

Precast Concrete

Precast concrete members used for social-cultural housing (in Rumanian)

M. Barbaiani, *Industria Constructiilor* and *Industria Materialelor de Constructii* (Bucharest), V. 7, No. 8, Aug. 1956, pp. 478-484

Reviewed by J. J. Polivka

Various types of pitched roof, floors and ceilings assembled of precast and prefabricated members are described and illustrated and their advantages and costs discussed.

Industrial concrete floors with thermal insulation (in Rumanian)

G. St. Georgescu, *Industria Constructiilor* and *Industria Materialelor de Constructii* (Bucharest), V. 7, No. 8, Aug. 1956, pp. 471-478

Reviewed by J. J. Polivka

Describes usual types of industrial floors and investigates their thermal insulation theoretically and on the basis of tests. Author proposes a new type of precast concrete floor assembled of units of reinforced concrete, 20 in. square, 2 in. thick, with ade-

quate corner supports resting on rough concrete underlayer. Continuous space under the top slab is filled with mineral wool or other insulating material. Thermal transmission coefficient of such floors is lower than that of foam concrete and only 35 percent of the value for ordinary concrete. The results of the research are recommended for use in building specifications.

Modifications in a precast-concrete works

The Engineer (London), V. 203, No. 5279, March 29, 1957, p. 496

Reviewed by Aron L. Mirsky

Brief description of modifications in the plant of the Modular Concrete Co., to permit casting of large and complicated structural members. Traveling skip employed is characterized by many automatic features.

Prefabricated roof structures for stables (in Hungarian)

L. Gabor, *Magyar Epitöipar*, V. 4, No. 12, 1955, pp. 538-545

Hungarian Technical Abstracts V. 8, No. 3, 1956

A description of prefabricated roof and ceiling structures designed for stables. The basic structural element in various combinations lends itself for roofs of various span and shape without intermediate support. Only three different elements have been designed (triangular girders, ceiling slabs, and supports for roofing material). The entire roof structure can be assembled from these three elements using simple dry joints throughout. The special requirements of stables in respect to heat insulation have been met. The method of erection (consecutive principal stages of assembly) is also illustrated.

Proceedings of international congress (1954, Dresden) for building with precast concrete (Die Montagebauweise mit Stahlbetonfertigteilen und ihre aktuellen Probleme)

Veb Verlag Technik, Berlin, 1956, 330 pp., 47 DM

One of the most impressive collections of papers on precast concrete construction available. The cloth bound book contains some 30 papers which were presented at the congress, together with the discussions and illustrations. About 600 pictures of actual construction of precast concrete structures, design drawings, detail drawings, forming, and erection are included. The precast applications discussed include precast shells, folded plates, trusses, arches, trussed arches, panels, rigid frames, and decorative grills for factories, crane bays, apartment buildings, large auditorium and exhibition halls, office buildings, and bridges. The excellence and number of illustrations and drawings make this book useful and valuable as a reference on precast concrete for all structural engineers, architects, precast concrete producers, and others interested in this field regardless of ability to translate German.

Heavy precast concrete structural work

The Engineer (London), V. 203, No. 5276, March 8, 1957, pp. 372-373

Reviewed by Aron L. Mirsky

Brief descriptions of three recent British structures using heavy precast elements: a factory at Stratton St. Margaret, a wind tunnel for the Ministry of Supply, and the Frodingham-Scunthorpe footbridge. Last used precast pretensioned main beams 67 to 86 ft in span and weighing 30 to 35 tons.

Prestressed Concrete

Prestressing with concentrated stress members (Vorspannung mit konzentrierten Spanngliedern)

Fritz Leonhardt and W. Baur, Wilhelm Ernst and Son, Berlin, 1956, 88 pp.

Reviewed by J. J. Polivka

An important addition to Leonhardt's *Spannbeton fur die Praxis* (published by Ernst in 1955), the book describes various methods used for the installation of stressing devices which consist basically of jacks inserted between two portions of concrete at each end of the structural member. The following characteristics and advantages of the improved Baur-Leonhardt method are presented: (1) Stress cables of a girder are more concentrated and preferably installed in a sheet-metal channel. (2) Cables of parallel wires or strands are arranged in several horizontal layers with small spacings allowing proper embedding in injected cement mortar. (3) The stressing cables are arranged in polygonal shape, the straight sections, 6 to 25 ft long, being interconnected by short channel sections in such a way that no fric-

tion can affect the post-tensioning. (4) These connections are replaced by curved sections in continuous girders over the supports. (5) If the angle between straight sections is sharper, special sliding sheets are inserted to decrease the friction. (6) Anchorage of cables in one or more channels is achieved simultaneously by winding them around reinforced concrete anchor blocks, and the cables are stressed all at once by hydraulic jacks inserted between these blocks and the girders, thus reducing the number of stressing operations.

(7) Such concentration of stressing members with special hydraulic jacks of high tension (300-500 tons) has the advantage that coarse concrete aggregates with high compressive strength and low plastic flow and shrinkage can be used. (8) Large anchor blocks and channel plates transmit the stress forces more uniformly into the girders. (9) Simultaneous stressing of greater cable group makes better control of stress intensity possible. Details and fabrication of slide channels in steel and in reinforced concrete are thoroughly discussed and illustrated and various types of anchor blocks and jacking devices described.

Special chapters refer to construction operations, especially installation of hydraulic presses, injection of fine cement mortar, measurement of stress intensities, concreting of the girder bodies. Of great interest is the list of 108 selected bridges built on the basis of these principles not only in Germany, but also in the USA, South America, Switzerland, Austria, and Norway from 1949 to 1956. Credit for these improvements is given also to W. Andrä.

Analysis of prestressed concrete structures and the application of recent research

PETER B. MORICE, *Proceedings*, Institution of Civil Engineers (London), V. 6 (session 1956-57), Mar. 1957, pp. 445-477 (discussion, pp. 477-492)
Reviewed by ARON L. MIRSKY

Paper, "in the nature of a general report on the recent research of the Cement and Concrete Association on the behaviour of prestressed concrete structures," covers much territory. Main subjects are continuous prestressed concrete frames, and strength of bridge decks under concentrated loads, subdivided under following headings: flexure of prestressed concrete sections, elastic design of prestressed concrete continuous beams and plane frames, applicability of continuity to prestressed beams; elastic analysis of highway bridges; ultimate strength of statically indeterminate frames, ultimate strength of bridge slabs; friction in prestressing tendons, transmission length in pretensioned members.

Prestressing systems (Voorspansystemen)

HANS WITTFOHT, *Cement* (Amsterdam), V. 8, No. 21-22, Oct. 1956, pp. 509-514
Reviewed by JOHN W. T. VAN ERP

The PZ prestressing cable consists of a bundle of wires each of which is of oval cross section and cross-ribbed. Cables are furnished in lengths of 1000 ft or more, in rings, which stretch themselves perfectly straight when unwound. Various anchorage systems for the cable are available. Many applications are shown in bridges, buildings, and industrial construction.

New development of prestressing concrete in France

M. S. WINIARSKI, *Civil Engineering and Public Works Review* (London), V. 52, No. 611, May 1957, pp. 553-556

Describes some new developments in prestressing plants in France, including use of tie-down anchors for pre-tensioned units in long-line production. Details of the equipment used are illustrated as well as some structural applications of the resulting products.

Construction method of a shell-type roof structure in prestressed concrete consisting of precast elements (in Rumanian)

A. SIRCHIS, *Industria Constructiilor* and *Industria Materialelor de Constructii* (Bucharest), V. 7, No. 8, Aug. 1956, pp. 461-469
Reviewed by J. J. POLIVKA

Roof covering a building 145 x 51 ft consists of ten barrel shells, spanning across the building with 14.32 m clearance, each barrel shell being assembled of ten sections supported by 12 x 20 in. prestressed concrete girders. These girders also carry longitudinal barrel shell along the center line of the building resting on skylight walls. Thickness of shells is 1½ in. Of special interest is the comparison with quantity of materials required for a typical reinforced concrete roof.

Stress concentration in post-tensioned prestressed concrete beams

GERALD PICKETT and K. T. SUNDARA RAJA IYENGAR, *Journal of Technology*, Bengal Engineering College (India), V. 1, No. 2, Dec. 1956, pp. 105-112
AUTHORS' SUMMARY

Use of the multiple Fourier method to analyze stress distribution in end regions of a post-tensioned prestressed concrete beam has been shown. The multiple Fourier method demonstrated here is a relatively new method for solving those problems for which the Saint Venant principle is not applicable. The actual three-dimensional problem and a two-dimensional simplified representation of it are treated.

Prestressed concrete designs

CARL L. BANKSTON, JR., *Consulting Engineer*, V. 9, No. 2, Feb. 1957, pp. 86-90, 92, 94
Reviewed by ARON L. MIRSKY

Discusses basic design concepts, with especial reference to Bureau of Public Roads' *Criteria for Prestressed Concrete Bridges*. As illustration, design of Wisner Drive Overpass, New Orleans, is analyzed and discussed. (Noteworthy is fact that measured live load stresses are far below theoretical values, indicating transverse distribution of loads is greater than allowed for in current AASHO specifications.) Economy of prestressed concrete is emphasized.

Properties of Concrete

Illinois develops high pressure air meter for determining air content of hardened concrete

J. D. LINDSAY, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 424-435 (including discussion)

Describes the development and use of an air meter for measuring air content of hardened concrete similar to that employed for plastic concrete except that the test pressure required is 5000 psi. The method appears to give good reproducibility and a good correlation with determination of air content in the plastic concrete and also by the microscopic examination of hardened concrete. The method may also be used to determine air content of plastic concrete. Experience in use is now extended to routine determinations of air contents of cores drilled from pavements for the last 2 years.

While 5 days are required for testing an individual specimen due to the necessity for drying and subsequent saturation, a large number of specimens can be handled quickly at the same time. Number of tests made has ranged as high as 65 per day.

Infra-red curing of concrete

Civil Engineering and Public Works Review (London), V. 51, No. 603, Sept. 1956, p. 981
HIGHWAY RESEARCH ABSTRACTS
Feb. 1957

A note on the application of infra-red radiation to the curing of concrete was published in the Russian periodical *Beton i Zhelezobeton*, July 1956, by B. V. Spektor. His results show that this new curing technique affects early concrete strength in a much shorter time than in the case of the usual steam treatment. It has been found that the maximum temperature of heat treatment should be 90 C. There is a loss of strength of concrete at temperatures below and above 90 C. Concrete subjected to the isothermal heat treatment for 2 to 4 hr will attain 50 to 70 percent of its designed strength. An admixture of calcium chloride to the infra-red cured concretes has much the same effect as in the case of steam curing; that is, additional strength.

Influence of water saturation on the tensile strength of hardened cements and concretes (in Russian)

M. Y. LESHCHINSKII, *Doklady Akademii Nauk SSSR*, V. 109, No. 3, 1956, pp. 521-523
CERAMIC ABSTRACTS
May 1957 (Kamich)

The tensile strength of cements and concretes and elongation in the course of bending, when saturated with water, are in complete agreement with the theory of the decrease in strength with adsorption.

Prediction of creep behavior in concrete from sonic properties

T. S. CHANG and C. E. KESLER, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 436-443

Proposes a method to relate the creep behavior of concrete in actual compression with its viscoelastic properties obtained by vibrating the concrete specimens at relatively small amplitudes. Concludes that a correlation does indeed exist between sonic and creep properties of concrete but that the limited

data in the paper will not support the analysis for application to other specimens, methods of curing, or other concretes, and that further research is needed to determine the general applicability of the procedure.

Study shows how prolonged mixing affects properties of concrete

Concrete Construction, V. 1, No. 1, Sept. 1956, p. 13
HIGHWAY RESEARCH ABSTRACTS
Jan. 1957

A report by the Bureau of Reclamation draws some interesting conclusions regarding the effect of prolonged mixing on various properties of concrete.

Prolonged mixing or delay up to 3 hr, without restoration of slump or air content, increased the compressive strength, modulus of elasticity, and resistance to freezing and thawing of the concrete.

Restoration of the slump and air content of air-entraining concrete after prolonged mixing up to 3 hr caused no reduction in strength or modulus of elasticity below concrete mixed for a normal period, but was detrimental to freezing and thawing resistance.

Electron emission and induced photoelectric effect on concrete cubes in compression (Exoelektronen und induzierter Photoeffekt bei Druckversuchen an Betonwuerfeln)

G. ULSENHEIMER, *Forschung auf dem Gebiete des Ingenieurwesens* (V.D.I., Duesseldorf), Vol. 23, No. 1-2, 1957, pp. 65-68
Reviewed by ARON L. MIRSKY

Tests indicated that concrete in compression emits electrons. Emission is a function of the applied load on the test cube, and of the degree of damage inflicted on the emanating surface by the load.

Paper is part of issue dedicated to Ludwig Föppl on his 70th birthday.

Investigation of electro-osmosis for reducing the water in concrete (in Russian)

Y. M. GRIGORIYA and L. A. OSIPOV, *Gidrotekhnicheskoe Stroitel'stvo*, V. 25, No. 10, 1956, pp. 15-18
CERAMIC ABSTRACTS
May 1957 (Kamich)

The amount of water which separates from concrete under the action of electro-osmosis increases with the voltage. Optimum voltage is 60 to 65 V and optimum current is 2 to 4 amp. The maximum amount of water removed was 13.4 percent of all the water used in mixing; this made it possible to reduce the water-cement ratio from 0.54 to 0.47. Observations during 1 hr showed that the water separated essentially during the first 5 to 7 min, after which the separation decreased, reaching zero. The strength of specimens subjected to osmosis was 35 percent higher than that of control specimens. A combination of vibration and electro-osmosis should result in greater separation of water.

Change in strength of concrete during its saturation with water (in Russian)

N. A. MOSHCHANSKII, *Gidrotekhnicheskoe Stroitel'stvo*, V. 25, No. 10, 1956, pp. 18-23
CERAMIC ABSTRACTS
May 1957 (Kamich)

Certain new experimental data on the strength of concrete in water are presented. Increase in strength of concrete under optimum conditions proceeds slowly (in decades) and steadily, gradually slowing down, approximately proportional to the logarithm of the time. The main causes for increase in strength are: gradual dehydration and packing of gel-like new formations (xerogels); and formation of crystalline concretions. Optimum conditions for the hardening of xerogels is moist air or water medium; dry air is unfavorable. Phenomena which take place in concrete during prolonged and deep saturation with water are more complex.

Increasing the frost resistance of steamed concrete for face plates (in Russian)

V. M. MEDVEDEV, N. A. VTOROV, and G. G. KOROLEV, *Gidrotekhnicheskoe Stroitel'stvo*, V. 25, No. 3, 1956, pp. 15-20
CERAMIC ABSTRACTS
May 1957 (Kamich)

Steamed concrete without the addition of sulfite cellulose wash water forms microcracks which have little effect on strength but which reduce frost resistance at an early age owing to the penetration and freezing of water. Addition of calcium chloride does not improve the quality of steamed concrete. Addition of sulfite alcohol wash water eliminates microcracks and raises frost resistance. Optimum steaming conditions are as follows: storage at 15 to 20 C for 4 hr, uniform increase in temperature for 6 hr, steaming at 75 ± 5 C for 6 to 8 hr, and uniform cooling in a moist medium to 20 C for 4 hr.

Osmotic method of increasing concrete strength

E. Pogany, *Civil Engineering and Public Works Review* (London), V. 51, No. 606, Dec. 1956, pp. 1349-1351
Highway Research Abstracts
May 1957

The author's investigations in 1952 and 1953 into the electro-osmotic method, showed that the strength of concrete could be increased by 35 to 45 percent at relatively low cost.

The advantages of electro-osmosis, which are not available in other methods, apply particularly in the following cases: (1) when a wet concrete is used, for example in the construction of thin walls of silos where all other methods applicable to plastic concrete only are unsuitable; (2) in the production of prefabricated elements and monoliths; (3) in winter, when the permissible temperature range for the process is +30 to 50 C without any cost increase.

Structural Research

Effect of axial load on the shear strength of reinforced concrete beams

M. J. Baron and C. P. Siess, *Civil Engineering Studies, Structural Research Series*, No. 121, University of Illinois, June 1956, 52 pp.
Highway Research Abstracts
Apr. 1957

Three modes of failure were observed: shear-compression, diagonal tension, and flexure. The shear-compression failures occurred in the shortest beams, the diagonal tension failures occurred in the medium length beams, and the flexural failures occurred in the two longest beams with the small steel percentage.

Bending stiffness of reinforced concrete beams: theory and experiments (in German)

K. Jager, *Osterreichische Bauzeitschrift* (Vienna), V. 11, No. 9, Sept. 1956, pp. 195-203
Applied Mechanics Reviews
May 1957 (Meyerhof)

Based on elastoplastic theory, author derives expressions for curvature of reinforced concrete beams under bending moment and hence obtains formulas for flexural rigidity of singly- and doubly-reinforced rectangular sections under working loads and at ultimate collapse. Theoretical relationships are compared with tests on some simply supported beams under third-point loading and fair agreement is found throughout the loading range.

Ultimate strength of reinforced concrete beams in shear

Royston Jones, *Magazine of Concrete Research* (London), V. 8, No. 23, Aug. 1956, pp. 69-84

Summarizes conclusions drawn from tests on a number of beams of identical dimensions but of different concrete strengths and from a re-assessment of tests on beams of various dimensions reported by Sunderland and Wilby. The limitations of recently advanced shear-compression moment theories (Laupa, Siess, Newmark, and Zwoyer) are demonstrated and a new expression is developed on similar principles.

Lateral pressures on retaining walls due to backfill surface loads

M. G. Spangler and Jack L. Mickle, *Bulletin* No. 141, Highway Research Board, Jan. 1956, pp. 1-19

Presents the results of measurements of the lateral pressure on full size retaining wall due to earth fill only and also due to concentrated uniformly distributed, or line load surcharges. Up to a 10-ft depth, lateral pressure due to backfill only followed the common assumption of a triangular pattern with $k = 0.33$. Lateral pressures due to surcharges closely approach those predicted by the Boussinesq theory of stress distribution.

An experimental investigation of the influence of the formation of a plastic hinge on the shear strength of a singly reinforced concrete beam

N. M. Khan and A. H. Mattock, *Magazine of Concrete Research* (London), V. 8, No. 24, Nov. 1956, pp. 151-156
Authors' Summary

A series of rectangular reinforced concrete beams was tested in such a way as to simulate the conditions at a plastic hinge over a support of a continuous beam approaching failure. With the plastic hinge moment constant, the shear force on the section was increased until failure occurred in shear. Similar sections were tested in shear in an uncracked state.

Results indicate that the presence of a plastic hinge does not reduce the ultimate shear strength of a reinforced concrete section.

Confirmation of inelastic stress distribution in concrete

EIVIND HOGNESTAD, *Proceedings*, ASCE, V. 83, ST 2, Mar. 1957, pp. 1189-1 to 1189-17

AUTHOR'S SUMMARY

Studies reported in the literature regarding inelastic stress distribution in flexure of reinforced members are reviewed, and the results obtained are compared to the recommendations of the ACI-ASCE committee on ultimate strength design. Test results show that the inelastic concrete stress distribution consists of a rising curve from zero to the maximum stress and a descending curve beyond the maximum stress. A simplified rectangular distribution gives a satisfactory accuracy for the common practical design cases. The ACI-ASCE committee design coefficients are well substantiated by test results.

Tests of the bond between concrete and steel

K. P. PEATTIE and J. A. POPE, *Civil Engineering and Public Works Review* (London), V. 51, 1956: No. 596, pp. 181-184; No. 597, pp. 314-316. *Building Science Abstracts*, V. 29, No. 9, Sept. 1956, Abstract 1473, p. 293

HIGHWAY RESEARCH ABSTRACTS May 1957

A theoretical analysis of the various stages of bond action between concrete and steel is followed by an account of torsion and pull-out tests on machined and polished bars. The tests were arranged so that the third or bearing stage of bond action did not exist. The adhesive stage of bond action terminates when a critical value of movement or strain in the concrete is attained. The distribution of load and bond stress along a bar in the adhesive stage is exponential in both the pull-out and torsion specimens. During the frictional stage, load distribution is dependent on the frictional force acting, the value of this force decreasing with increasing amounts of relative movement between the two materials.

Contributions to the study of steel anchorage in prestressed concrete (in Rumanian)

M. HALMAGIU and L. SCRIPCA, *Industria Constructiilor* and *Industria Materialelor de Constructii* (Bucharest), V. 7, No. 8, Aug. 1956, pp. 450-460

Reviewed by J. J. POLIVKA

Results of laboratory tests with Korovkin anchorage system are presented, and this system is compared with other systems (Freyssinet, Magnel, Leonhardt-Baur). Advantages and disadvantages of all these systems are thoroughly discussed.

General

A new soniscope—"The Elastiscope"

D. R. LAMB, *Proceedings*, Highway Research Board, V. 35, 1956, pp. 418-423

A simplified type of soniscope apparatus which can be used to measure the velocity of a supersonic wave through 1 to 5 ft of concrete. Limited to discussion of the equipment rather than results achieved with it. Includes circuit diagrams, schematic diagram, and description of the subassemblies employed.

Neutron moisture meter for concrete

J. PAWLIW and J. W. T. SPINKS, *Canadian Journal of Technology* (Ottawa), V. 34, No. 8, Mar. 1957, pp. 503-513

Reviewed by ARON L. MIRSKY

Describes development and calibration of a portable moisture meter, using a source of fast neutrons and a slow-neutron detector. Apparatus affords rapid and accurate nondestructive measurement of total moisture content of a concrete member; safety of operator has not been neglected.

Lectures—1954 annual convention of the German Concrete Association (Vortraege-Hauptversammlung, Deutscher Beton-Verein EV, 1954)

Deutscher Beton-Verein EV, Wiesbaden, 1954,* 354 pp.

Reviewed by ARON L. MIRSKY

Volume contains 13 technical papers, all in German, which afford the reader an excellent survey of what is being done with (and to!) concrete today:

Boundaries of the specialties

EUGEN WAHL, pp. 74-85

Discussion of the role of the specialists: testing engineers, soils engineers, etc., in construction.

Development, current status and possibilities of concrete dams

JEAN-PIERRE FREY, pp. 86-107

Discusses materials, construction; pure gravity-type, gravity-arch, pure arch-type dams, and concludes with a few comments on war damage to dams.

*The proceedings of the 1955 convention were reviewed in "Current Reviews," ACI JOURNAL, Nov. 1956, *Proc.* V. 53, p. 560.

Structures using precast members of reinforced concrete and prestressed concrete in Great Britain.
KALMAN HAJNAL-KONYI, pp. 108-143

Brief descriptions of many British buildings and bridges.

Prestressed shell-type structures
ULRICH FINSTERWALDER, pp. 145-161

Descriptions of various structures of this type, with a brief discussion of the principles. An abstract of this paper, by H. Seidel, appeared in *VDI Zeitschrift*, V. 97, No. 8, Mar. 11, 1955, pp. 249-250.

Model investigations and skewed slabs
GOTTHARD FRANZ, pp. 166-185; discussion, pp. 185-187

Technical details of experimental investigation of a slab spanning over two unequal spans, with a 71-deg skew angle. Investigation has also been described in *Der Bauingenieur*, May 1954, pp. 182-190 ("Current Reviews," ACI JOURNAL, Feb. 1956, *Proc.* V.52, p. 690).

Investigation of vibration of prestressed structures
HERMANN BAY, pp. 189-203

Two structures were investigated: a bridge and a grandstand. Correspondence between theoretical and measured frequencies was quite good.

Special problems in the construction of the Birsfelden power plant
ARMIN AEGERTER, pp. 205-228

Layout, foundation, construction, and similar topics of this low-head plant are treated.

Concrete streets and (their) bases
LOTHAR SCHAIBLE, pp. 229-250; discussion, pp. 250-253

Frost damage and how to prevent it.

Newer Swedish slip-form jobs
ULF WIDERSTROM, pp. 255-261

Silos, a chimney, a retaining wall are described.

Newer slip-form structures in Germany
GUSTAV MERKLE, pp. 262-279

Silos, a rectangular bunker, buildings.

Newer hydropower structures on the Inn and the Donau
WILHELM MOCKERT, pp. 280-305

Primarily on construction plant.

Construction with precast reinforced concrete elements
ERNST LEWICKI, pp. 307-323

Describes an East German conveyor bridge, a sports arena utilizing three-hinged arches, Hungarian "halls" utilizing rigid frames with trussed transoms and Vierendeel-type columns, a salt-storage shed using three-hinged arches (the method of erection of the arches and the erection stresses are of especial interest), and other structures.

Experiences with concrete additives, especially air-entraining agents
FRITZ WEBER, pp. 325-340; discussion, pp. 340-350

Discusses workability-improvers (plastifiers), densifiers, air-entraining agents.

Heat requirements of snow melting systems

W. P. CHAPMAN and SAMUEL KATUNICH, *ASHAE Journal Section, Heating, Piping and Air Conditioning*, Feb. 1956, pp. 149-153

Presents an extended theory for thermodynamic design of snow melting systems. Theoretical results are compared to observations. Includes a table to simplify application of the data for practical use, derivation of equations, and examples.

Concrete research in progress in the northern countries (in Swedish)

Nordisk Betong (Stockholm), V. 1, No. 1, 1957, pp. 89-108

Reviewed by MARGARET CORBIN

An annotated bibliography.

Structures in modern architecture (La struttura dell'architettura moderna)

A. ARCANGELI, Sansoni Edizioni Scientifiche, STEB, Bologna, 1956, 370 pp., 4000 lira

Reviewed by J. J. POLIVKA

This book, presenting all basic principles of structures used in modern architectural design, is being used as textbook in the College of Architecture, of Florence University. First chapter deals with structural materials and their mechanical characteristics; Chapter 2 with their behavior in completed structures; Chapter 3 deals with their deterioration and protection. The subjects of the additional chapters are: actual viewpoints about their elastic and plastic resistance; soil mechanics, foundations, their statics and disturbances (geotechnics); earthquake-proof structures; structural skeletons and schemes; prestressed concrete (precast and

hyperstatic structures), prestressed steel structures; recent developments of structural types (vacuum concrete, composite structures, floors in reinforced concrete combined with light steel skeleton; modern techniques in steel structures, welding, prefabricated light structural elements (extruded), particular criteria of their design, two- and three-dimensional structures, torsional effect, elastic instability; modern techniques of timber structures (various connections with bolts and nails, glue-lamination); prefabrication and specifications. Recent progress is thoroughly discussed and illustrated (shell structures, hyperbolic paraboloids, Chelazzi's "suspenarch" structures, etc.).

Winter concreting—Theory and practice

Proceedings, RILEM Symposium on Winter Concreting, published by Danish National Institute of Building Research, Copenhagen, 1956, 1574 pp., 65 D.kr. (deluxe binding 75 D.kr.)

The knowledge and experience is presented in two main parts, theoretical and practical. Book concludes with extensive conclusions prepared by the Danish National Institute of Building Research after careful study of all the papers and it is believed that these conclusions (which are in English, French, and German) will be important.

The first theoretical session dealt with the climatic conditions and their reproduction in the laboratory. Quantitative methods for prediction of heat conditions of the concrete are developed, but may need refinement and modifications.

The session dealing with laboratory experiments for determination of resistance of concrete to early freezing concentrated on the problem of "prehardening," i.e., the determination of that period prior to the hardening proper, in which the concrete will be definitely damaged if exposed to freezing temperatures. Different prehardening times were found by different investigators, but, as it was also discussed, prehardening is not a function of time alone and should be considered dependent on several concrete qualities. A safe estimate of prehardening time can be deduced.

The next session treated the hardening process as influenced by temperature. The papers and discussion presented made it evident that the hardening process with sufficient accuracy for practice could be expressed by a time-temperature function.

The last theoretical session covered resistance of concrete to frost at early ages. Results of basic research were presented by T. C. Powers. The elucidation on the freezing mechanism in hardening concrete gives a profound understanding of the unsolved problems of the two previous sessions. The effect of air entrainment and means to estimate a safety limit (prehardening) for green concrete were focal points of this session.

The first practical session treated the question of how to obtain adequate concrete quality under winter conditions. The practical procedure of winter concreting is discussed in general, especially to what extent and under which conditions specific measures should be taken, as elucidated by practical examples.

The last session dealt with procedure and equipment on the building site. Here many experiences and methods of practice are discussed and interesting information presented. Comparison between efficiency and economics of the various possibilities seems a promising field for further progress.

Comparison of standard specifications for concrete structures in Denmark, Finland, Norway and Sweden (in Swedish)

Viljo Kuuskowski, *Nordisk Betong* (Stockholm), V. 1, No. 1, 1957, pp. 37-55

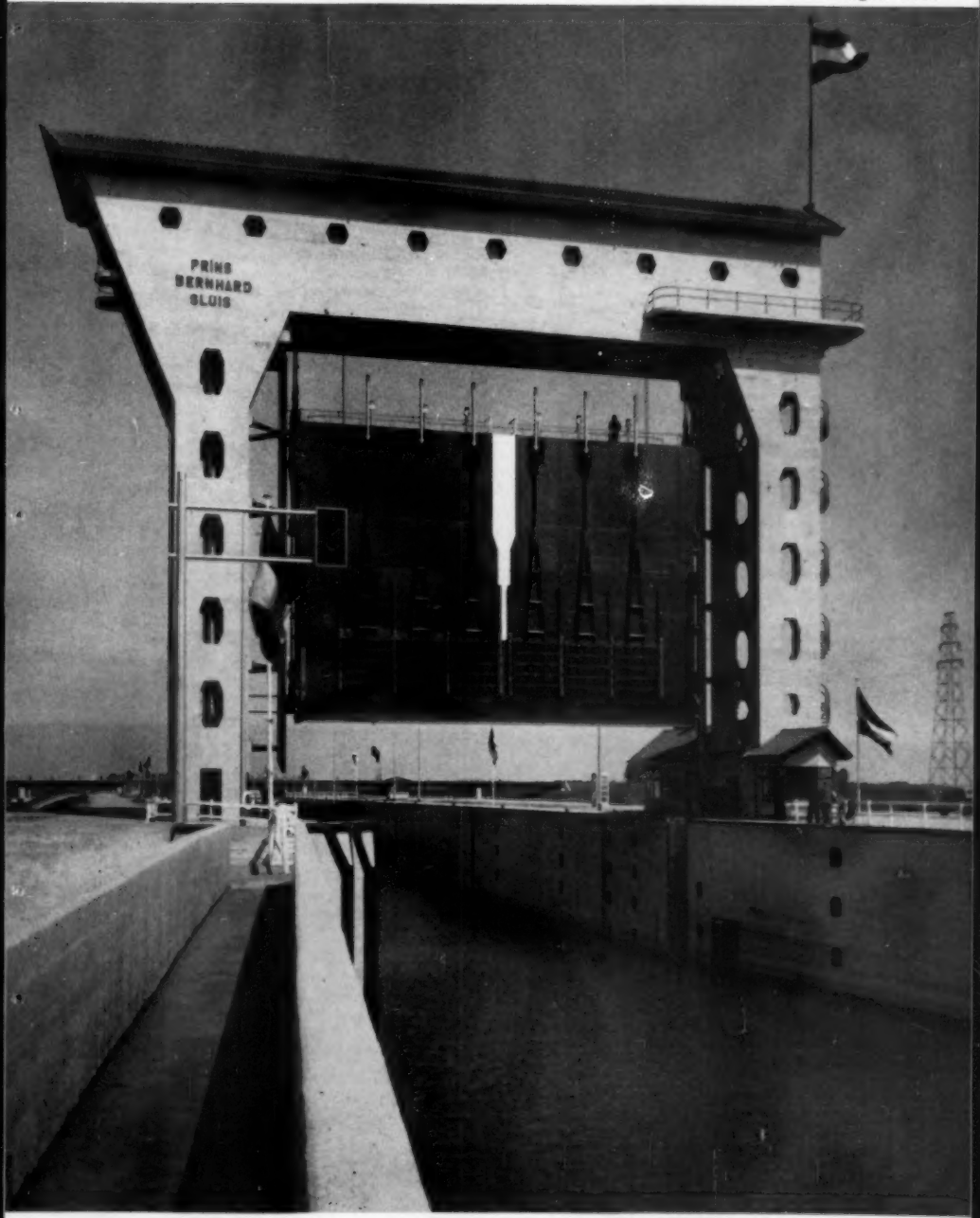
Reviewed by Margaret Corbin

Describes standard specifications for plain and reinforced concrete structures in Denmark, Finland, Norway, and Sweden and discusses possibilities of adopting joint standard specifications in these countries. Author recommends that these specifications should be divided into two parts: the "legal rules," and the "technical instructions," in the same way as the present Finnish standards. The legal rules should state the fundamental principles, and should be common to all these countries. In addition, each country should issue technical instructions adapted to local conditions. Writer believes that the stipulation of a minimum cement content in the concrete is an obstacle to progress.

Concrete is divided into classes according to quality: two in Denmark and Norway, three in Finland and Sweden. Strength calculations should to an increasing extent be made by means of the ultimate load methods.

ACI News Letter

Vol. 29, No. 2 JOURNAL of the AMERICAN CONCRETE INSTITUTE August 1957



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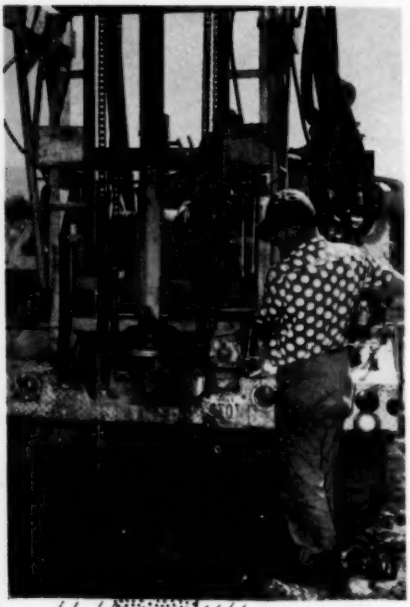
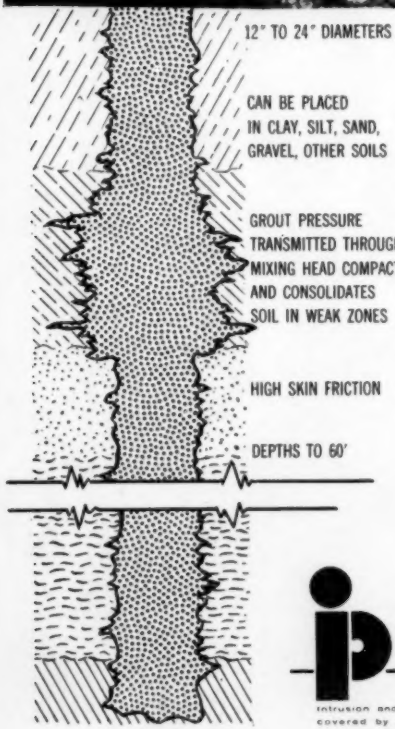


NO VIBRATION OR GROUND HEAVING— Truck-mounted mixing equipment places versatile Mixed-in-Place piles without removing material. Piles can be placed in many sand, gravels and rock-filled soils not penetrable by driven piling.

economical cut-off walls made with mixed-in-place* piles


12" TO 24" DIAMETERS
CAN BE PLACED IN CLAY, SILT, SAND, GRAVEL, OTHER SOILS
GROUT PRESSURE TRANSMITTED THROUGH MIXING HEAD COMPACTS AND CONSOLIDATES SOIL IN WEAK ZONES
HIGH SKIN FRICTION
DEPTHS TO 60'

Mixed-in-Place piles, a patented INTRUSION-PREPAKT technique, can be placed with overlapping elements to form a tight, sub-surface water cut-off. This cost-saving method was recently used instead of sheet piling for cut-off walls under the spillway sections of Slaterville Dam, Ogden, Utah and Putah Diversion Dam, Winters, California.

Originally developed and widely used for underpinning, bearing piles and soil stabilization, Mixed-in-Place piles are placed without the vibration commonly associated with driven piling. Highly mobile equipment mixes INTRUSION grout with in-place soil to produce a pile of excellent load bearing capacity.

When overlapped, Mixed-in-Place piles make effective shoring walls for construction or water-tight cut-offs and cofferdams. This often eliminates excavation, shoring, dewatering and backfilling. Mixed-in-Place piles in diameters from 12 to 24 inches can be placed at depths to 60 feet.

For further information,
contact INTRUSION-PREPAKT, INC., Room 568-H,
Union Commerce Bldg., Cleveland 14, Ohio.
In Canada: INTRUSION-PREPAKT, LTD.,
159 Bay Street, Toronto, Ontario.

*U.S. Patent Nos. 2,313,107; 2,655,004; 2,782,605 and Patents Pending

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ACI

News Letter Contents

August 1957

On the Cover—Prince Bernhard Sluice at Tiel, The Netherlands, built 1948-52, can be used by ships up to 4,300 tons for inland navigation. The vibrated concrete was finished with a mixture of one part cement to three parts of silver sand. Sluice chamber is 350 m long and 18 m wide. Architect and engineer for the structure was J. P. Josephus Jitta. Contractors were Nederlandse Aannemers-Aanneming Mij. v/h H. F. Boersma and Aannemersbedrijf F Dirk Verstoep, The Hague.

—*Photograph courtesy* Cement, *Amsterdam*

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Floor installation under way at

ACI BUILDING

ACI'S NEW HOME in northwest Detroit progresses toward its occupancy date of January 1958—floor construction is the most recently completed aspect in the unfolding of Minoru Yamasaki's unique design in concrete.

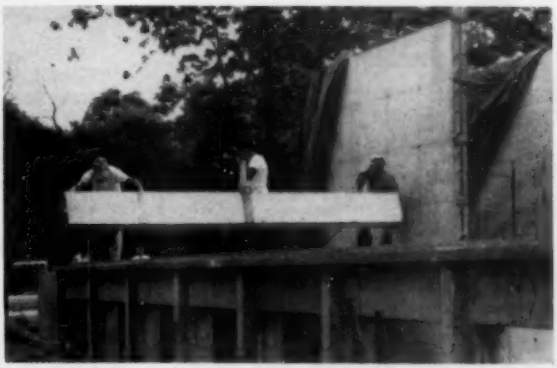
Basement floors were cast over a base course of compacted 4-in. bank run gravel and a 4-mil plastic vapor barrier. The 4-in. slab was reinforced with wire mesh, and will have a 2-in. topping of concrete placed before the floor covering materials are added.

Precast floor units

Hollow precast units manufactured by Price Brothers Co., Michigan Flexicore Division, Livonia, Mich., have been used for all of the main floor except cast-in-place corridor floors. Heating, ventilating, and wiring installations are made in the hollow core of the floor members. A 2-in. topping—made with 1 part cement, 2.5 parts sand, and 2.5 parts pea gravel—will be used above the precast floor units.

Footings and base for the free-standing chimney of the heating plant have been prepared at the back of the building.

Workmen guide Flexicore panel into position at ACI headquarters construction site in northwest Detroit



TAC announces appointments to Committee 201

The Technical Activities Committee has approved the appointments of the following members to serve on the recently organized ACI Committee 201, Durability of Concrete in Service. Hubert Woods, Director of Research, Portland Cement Association, Chicago, is chairman of the committee.

Robert F. Adams
California Department of
Water Resources
Sacramento, Calif.

Harold Allen
Bureau of Public Roads
Washington, D. C.

Delmar L. Bloem
National Sand and Gravel Association
and National Ready Mixed Concrete
Association
Washington, D. C.

Bruce E. Foster
National Bureau of Standards
Washington, D. C.

Thomas B. Kennedy
Waterways Experiment Station
Jackson, Miss.

Donald W. Lewis
National Slag Association
Washington, D. C.

W. J. McCoy
Lehigh Portland Cement Co.
Coplay, Pa.

P. D. Miesenhelder
Association of American Railroads
Chicago, Illinois

Walter H. Price
U. S. Bureau of Reclamation
Denver, Colo.

Charles H. Scholer
Kansas State College
Manhattan, Kan.

Bailey Tremper
California Division of Highways
Sacramento, Calif.

R. P. Vellines
Universal Atlas Cement Co.
Gary, Indiana

ACI technical committee appointments

The following members have been appointed to serve on the technical committees listed below; included are *new* appointments only.

Committee 314, Rigid Frame Bridges

A. Amirikian
Bureau of Yards & Docks
U. S. Navy
Washington, D. C.

Charles Birnstiel
New York University
New York, N. Y.

Anton Tedesko
Roberts and Schaefer Co.
New York, N. Y.

H. B. Zackrison, Sr.
Office, Chief of Engineers
Department of the Army
Washington, D. C.

Committee 323, Prestressed Reinforced Concrete (Joint ACI-ASCE)

W. Burr Bennett, Jr.
Portland Cement Association
Chicago, Ill.

Committee 334, Concrete Shell Structures

Giorgio Baroni
Roberts and Schaefer Co.
New York, N. Y.

Wilhelm Flügge
Stanford University
Stanford, Calif.

Otto Gruenwald
Consulting Engineer
New York, N. Y.

Milo S. Ketchum, Jr.
Ketchum & Konkel
Denver, Colo.

Eric C. Molke
Prestressing Research and Development,
Inc.
San Antonio, Tex.

Alfred L. Parme
Portland Cement Association
Chicago, Ill.

Continued on p.

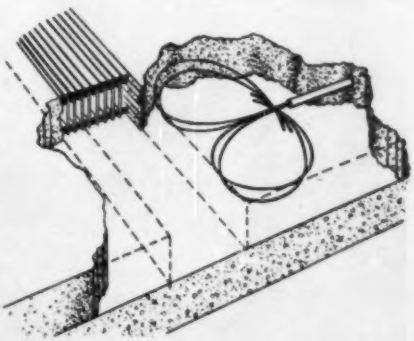
prestressed pavement

600-ft experimental roadway is now being tested in Pittsburgh

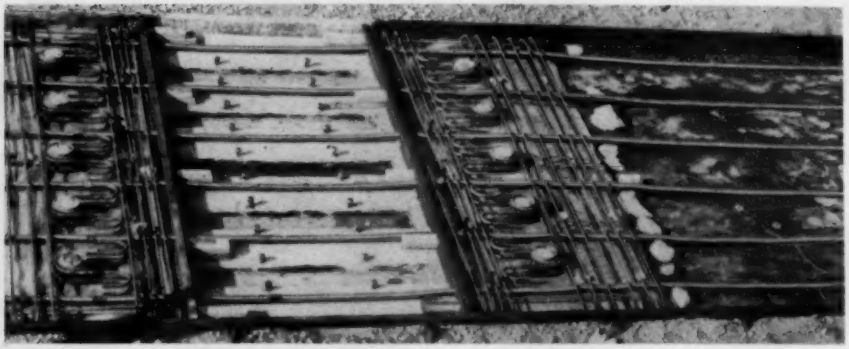
Six hundred ft of experimental prestressed highway has been built in Pittsburgh by Jones & Laughlin Steel Corp. Prestressing was done in February, and extensive tests on the roadway are now in progress (see photo above).

Connecting wire strands were anchored at opposite ends of a 400-ft section of roadway to be formed. The strands pass through flexible steel conduits embedded in the 5-in. concrete slab. When concrete was placed, a 6-ft gap was left in the middle of the 400-ft section. Hydraulic jacks were placed longitudinally in the gap. When the concrete had hardened, the gap was jacked apart to a width of 8 ft, far enough to produce the required tension in the wire strands, and consequent compression in the concrete.

When this was accomplished, the jacks were removed and concrete was placed in the gap. A special device was developed to hold the sections apart following removal of jacks and until the concrete hardened in the gap. Thus the entire 400-ft section of



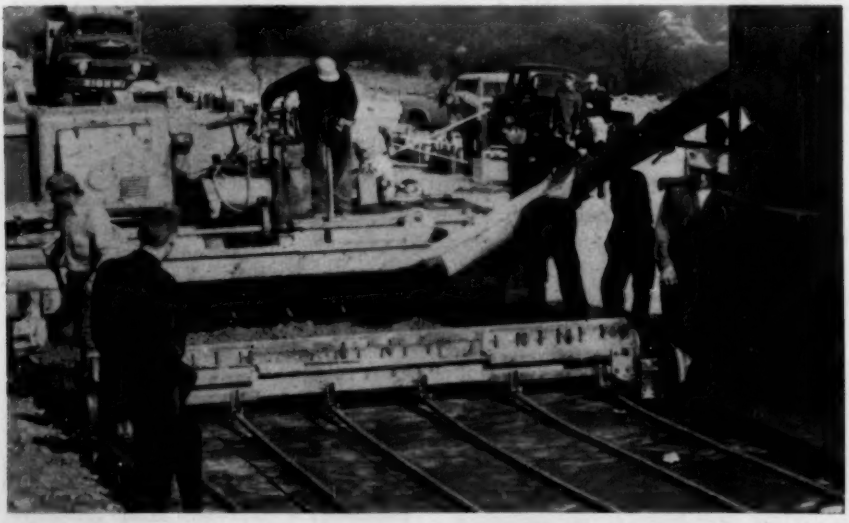
Steel strands emerge from flexible conduit at end of slab, form "butterfly" anchorage in concrete. Sketch also shows cut-away view of rubber and metal expansion joint 12 in. wide between slabs



Center of the 400-ft prestressed slab. Initially all concrete was placed except in the 6-ft gap. After concrete had set, jacks were placed in the gap and the two sections spread apart to a width of 8 ft to supply prestressing. Then a specially designed frame was placed in position to hold the two end sections apart, the jacks were removed, and concrete was placed in the gap. The five circular elements on either side of the gap, surrounded by special anchorage steel, were used as mounting points for the frame which maintained the expanded gap after jacks were removed

concrete was loaded in compression as a result of the tension in the embedded steel strands.

In addition to the main 400-ft section of pavement, a 100-ft prestressed section has been placed at either end. Joints in the experimental road, which will have to compensate for expansion and contraction of 400 ft of concrete, are 12 in. wide and extend the full depth of the concrete slab. Joint design allows the top surface to remain level during maximum expansion and contraction of the concrete sections. Developed by B. F. Goodrich Industrial Products Co., the joint



Wheeled device, which moved ahead of the paving machine, positioned tendons at proper elevation in the freshly placed concrete

includes a series of metal plates bonded to rubber to carry the vertical loads that traffic will impose on the joint.

The idea for the experimental roadway was suggested by Admiral Ben Morreell, chairman of the board of Jones and Laughlin, a past president of the American Concrete Institute, and long an authority on concrete and reinforced concrete. J. E. Morris, director of product development and John J. Murfay, development engineer, both of Jones & Laughlin have been in charge of the project. Engineering work was done by Richardson, Gordon & Associates, Pittsburgh.

Allegheny Contracting Industries, Inc., Pittsburgh, was responsible for construction work. Test specimens were studied at Carnegie Institute of Technology under the direction of Charles F. Peck. Instrumentation for testing of the prestressed section was designed by the Fritz Laboratory at Lehigh University, under the direction of W. J. Eney.

Continued from p. 4

D. A. Polychrone
Georgia Institute of Technology
Atlanta, Ga.

Mario G. Salvadori
Columbia University
New York, N. Y.

John B. Skilling
Worthington and Skilling
Seattle, Wash.

Robert Zaborowski
Roberts and Schaefer Co.
Chicago, Ill.

Committee 611, Inspection of Concrete

Frank R. Killinger
A. J. Hales Testing Laboratories
Oakland, Calif.

Committee 622, Formwork for Concrete

John Banker
Portland Cement Association
Chicago, Ill.

Committee 716, High-Pressure Steam Curing

David Watstein
National Bureau of Standards
Washington, D. C.

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Perlite Fireproofing

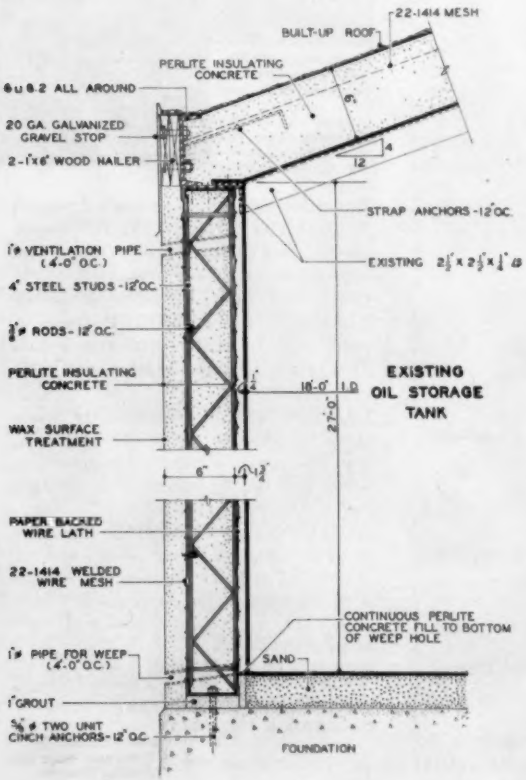
A technique has been developed for protecting gasoline and oil storage tanks from the dangers of fire and evaporation loss with perlite insulating concrete. United States Gypsum Co., Chicago, recently used the method on three medium-size fuel tanks at its Philadelphia plant. The company wanted to enlarge a building close to the 50,000-gal. fuel tanks but was unable to do so because of municipal fire regulations. Approval was granted, however, after the tanks had been fireproofed with perlite concrete.

Steel framework

A 6-in. thickness of perlite concrete was machine-sprayed onto a paper-backed wire lath which had been attached to a Trussteel stud framework erected around the tanks.

This framework was connected only at the top of each tank and to runners circling the base. Thus the perlite concrete protection was built structurally independent from the metal tank exterior so as to provide a 1¾ in. dead-air space. The insulating property of the perlite concrete barrier together with the dead air space is calculated as having a heat transmission value of 0.16 Btu per hr, per sq ft, per deg F of temperature difference from air to air in the given building section. This insulation value is equivalent to that of more than 3 ft of structural concrete.

Continued on p. 10



Cross section shows fireproofing details for 50,000-gal. fuel tank

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are completed sooner with

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On your next bridge or overpass, design for rugged beauty plus economy . . . design for REINFORCED CONCRETE.

A southerly view of a recently completed section of the Harbor Freeway, U. S. Highway 6, Los Angeles

Designed by the California Division of Highways

Contractor: Winston Bros., Monrovia California

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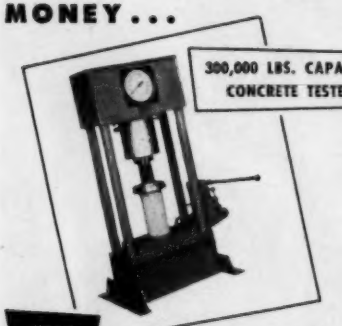

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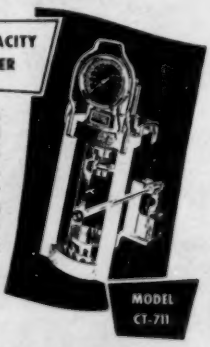

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Perlite . . .

Continued from p. 8

When completed, the perlite concrete was waterproofed with a wax preparation. Tank tops were also protected with perlite concrete and then covered with four-ply membrane gravel roofs.

Concrete mix

The composition of the concrete mix was as follows: 4 cu ft of perlite, one bag of portland cement, 5 lb Lumnite cement, 3 oz of NVX air entraining agent (vinsol resin solution) and 62-64 lb water.

Fire resistance

Perlite concrete sprayed walls of this construction are approved for 4-hr retardant ratings. They may be applied by any experienced lathing and plastering contractor equipped with spraying equipment.

The fire-retardant quality of perlite concrete is due, in part, to the fact that perlite concrete when exposed to fire releases chemically-combined water in the form of vapor. This action limits the temperature to 212 F until all of the water has been driven off. Also concrete made with perlite instead of heavy aggregate enjoys limited expansion at high temperatures. Consequently, there is a minimum of cracking upon exposure to flames, preventing fire from penetrating the concrete barrier.

Moorman to head civil engineering department

Robert B. B. Moorman, professor and chairman of the department of civil engineering at Syracuse University, has been appointed head of the civil engineering department at the Polytechnic Institute of Brooklyn.

Laughlin forms new firm

W. H. M. Laughlin, consulting engineer, formerly a firm member of Proctor, Redfern & Laughlin, has formed a new company, Laughlin, Wyllie & Ufnal, Toronto.

As senior engineers in the former firm Mr. Wyllie and Mr. Ufnal have been business associates of Mr. Laughlin for the past 10 years. The newly organized company will specialize in planning, design and supervision of industrial plants, buildings, bridges and other diversified civil engineering work.

ACI observer views

Atomic Test "Priscilla"

DOUGLAS McHENRY, REPRESENTING the American Concrete Institute in his capacity as vice-president, was one of a group of 26 special observers who witnessed the "Priscilla" atomic test shot detonated at the Atomic Energy Commission's Nevada test site near Las Vegas, June 24, 1957.

McHenry

The Priscilla shot was the fifth of the summer test series and included tests of Federal Civil Defense Administration shelters. Although nine shots in the current test series are "open shots" to which the press and groups of civilian observers are invited, the Priscilla shot was not open to the press. Only observers with special technical interests were invited as guests of the Federal Civil Defense Administration.

Family-type and industrial shelters have been tested previously, and additional tests of family-type shelters are scheduled during the 1957 series. However, there have been no previous tests under an actual atomic detonation of structures which could be used as community shelters.

Two types of structures proposed by engineers as feasible for a mass shelter program were subjected to the Priscilla blast. One of these was the reinforced concrete dome type; the other was a dual-purpose structure which could be used as an underground garage and converted to use as a shelter. A number of similar dual-purpose mass shelters have been constructed in Scandinavian nations.

Three reinforced concrete domes of 50-ft diameter and 6-in. constant shell thickness were constructed in Frenchman's Flat at the test site and were subjected to blast pressures ranging from 20 to 75 psi. Although dome shelters could be constructed either above or below ground, all of the test structures were exposed to blast without the aid of earth cover.

The test structures were considerably smaller than the dome shelters which have been advocated in some engineering circles. Plans have been advanced for dome structures up to 150 ft or more in diameter, but FCDA engineers decided that the 50-ft size was adequate for the preliminary tests.

The dual purpose underground shelter was approximately 90 ft square and was designed as a segment of a reinforced concrete structure which could be expanded to any desired size. It was under 3 ft of earth, with a reinforced concrete roof slab 2 ft 6 in. thick. Access to this shelter was by means of an auto ramp, leading down to a reinforced concrete door. This door, 4 ft thick and weighing approximately 100 tons, was mounted on a monorail.

Three reinforced concrete home type shelters, designed to protect a family of approximately six persons against nuclear blast and radiation, were also tested at various distances from ground zero. These shelters could be expected to provide minimum living conditions for as many as 30 persons.

All of the shelters were equipped with instruments to measure their reactions to nuclear blast and with neutron and gamma ray dosimeters to measure radiation.

The group of civil defense observers inspected the test structures on June 22, observed the detonation on June 24, and on June 26 (at zero plus 51 hr) were permitted a rather hurried expedition through the still slightly "hot" test area to view the damage.

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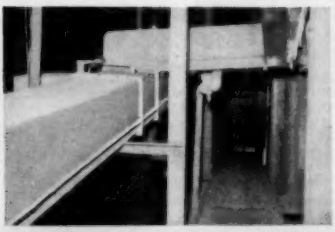

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Positions and Projects

Slip-form method, precast members used in church construction

Completion of a 75-ft tower 14 ft square with 200 tons of concrete—placing time 69.5 hr—drew hundreds of architects, engineers and construction personnel to inspect the

Swedish slip-form process used at the site of Redeemer Lutheran Church under construction in South Gate, Calif.

The church was designed by architects Robert Ainsworth and Edwin Westberg of Pasadena. Main framework of the building is formed by nine precast concrete arches. Some of the precast wall panels will weigh in excess of 25 tons. The roof will be formed by 10 precast panels between each of the nine arches. Over 2400 tons of concrete will go into the structure, according to C. J. Pankow, in charge of the building department of Peter Kiewit Sons' Co., the contractors.

Use of the Swedish technique saved approximately 25 percent in tower cost according to Pankow. For placing the tower, the slip form was raised 1 in. every 5 min, continuously until the job was completed. Four cu yd of concrete was in the form at all times, and there was about 2 tons form friction, but eight electrically operated jacks exerted more than adequate lifting force of 32 tons to raise the slip form.

ASTM honors Dresser and Voss

Twelve technical leaders in the field of engineering materials—men who have rendered outstanding service to the American Society for Testing Materials, particularly in its technical committee work—were honored during the ASTM 60th annual meeting when they received Awards of Merit.

Among those honored were ACI members, Theodore Parker Dresser, Jr. and Walter Charles Voss. Mr. Dresser, chief engineer and vice-president of Abbot A. Hanks, Inc., San Francisco, received the award in recognition of long-time valued service in advancing the interest of ASTM on the west coast, especially through Northern California District activities, and for support of technical and administrative work. Voss, professor emeritus and lecturer, department of building engineering and construction, Massachusetts Institute of Technology, Cambridge, was cited in recognition of loyal, long-time support of ASTM research and standards work, including both administrative and technical aspects, especially in ASTM Committee C-7 on Lime, which he headed for 10 years.

AASHO road test objectives cited

Objectives of the AASHO Road Test at Ottawa, Ill., were announced recently at the annual meeting of the Division of Engineering and Industrial Research of the National Academy of Sciences, National Research Council at Washington, D. C.

The announcement was made by Prof. K. B. Woods, civil engineering head at Purdue University and immediate past chairman of the Highway Research Board, a unit of the Division of Engineering and Industrial Research. The Highway Research Board is administering and supervising the road test, which is sponsored by the American Association of State Highway Officials. The AASHO Road Test is one of several investi-

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gations under way in specific areas under a mandate from Congress contained in the Federal-Aid Highway Act of 1956. Professor Woods pointed out that these short-term studies will contribute greatly to solving pressing highway problems, and will also form the basis for planning additional research efforts for the 1960 decade.

Professor Woods described the road test as a $20 million experiment where rigid and flexible pavements of "controlled cross section design" are being placed on bases and subbases of specified depth, and will be subjected to controlled traffic for two years. Axle loadings on the test vehicles will range from 2000 to 30,000 lb on a single axle and 24,000 to 48,000 lb on a tandem axle. A system of electronic instruments will record the effects on the pavements.

The general objective of the road test is to compile data on how pavement structures and bridges of various engineering standards are affected by motor vehicles of different sizes, weights, and axle-loading characteristics. Studies will be conducted on special traffic lanes made up of various thicknesses of base and subbase and various surface thicknesses of asphaltic concrete, plain portland cement concrete, and reinforced portland cement concrete. Special instruments will count the load applications and keep records of the behavior and condition of the test sections continuously during all seasons. Similar studies will be made on bridges of various designs to be built into the test loops.

Paul D. Smith to head Intrusion-Prepakt

The board of directors of Intrusion-Prepakt, Inc. has announced the election of Paul D. Smith to the office of president. The company's headquarters are in Cleveland.

Ivan M. Viest joins AASHO road test

Ivan M. Viest, formerly with the department of theoretical and applied mechanics at the University of Illinois, has taken a new position as bridge research engineer with the American Association of State Highway Officials road test at Ottawa, Ill. This is a project of the National Academy of Sciences administered through the Highway Research Board.

Mr. Viest reports that they are now in the process of building 16 bridges of various types designed as test structures. Four bridges are cast-in-place T-beam structures, and four are prestressed concrete beam structures.

Stressteel announces new officers

Stressteel Corp., Wilkes-Barre, Pa., has announced the election of Edward Schechter, to succeed Curzon Dobell as president. J. Nelson Hicks was reelected sales vice-president and chief engineer. Mr. Dobell was reelected as a member of the board of directors.

Ferguson appointed Intrusion-Prepakt regional manager

C. H. Ferguson was recently appointed manager of the San Francisco regional office of Intrusion-Prepakt, Inc., specialists in construction and maintenance of heavy concrete structures and foundations. Mr. Ferguson joined Intrusion-Prepakt in 1952. Previously he had served six years with the U. S. Army Corps of Engineers and on the Detroit Dam construction job in Oregon.

Bickel to head Southwest Concrete Materials Corporation

Southwest Concrete Materials Corp., Little Rock, Ark., announces the appointment of George Bickel as president and W. F. Tracy as vice-president.

Mr. Bickel pioneered the use of lightweight structural aggregate in the Dallas and Texas area. He also led in the development of the use of lightweight precast pretensioned concrete units in Texas. Mr. Bickel, a member of ACI, was one of the organizers of the Expanded Shale Institute and served as its secretary for 2 years.

AIA plans research activity

Preliminary plans for organization of a full-time research staff and a research program in architecture have been announced by the Committee on Research of the American Institute of Architects, Washington, D. C., in a statement issued by AIA as *Special Report No. 4*. Architectural research, according to the committee, deals with problems of function and form in buildings and their surroundings, and is thereby research in planning and esthetics. Solving these broad problems requires collaboration between the architectural profession and existing associations within the building industry.

The special report discusses need for documentation and dissemination of research results. It also cites four phases of research activity for the architect: (1) research within the practice of architecture, performed on problems arising from day-to-day working requirements; (2) identification of new problems in building deserving and requiring research; (3) research in schools of architecture, and (4) stimulation of research and development among other elements of the building industry.

Information index

A non-profit, self-supporting index of architectural and building information is now being developed as a service to which architects and others may subscribe. The service will include references to architectural and building knowledge already authenticated, published or unpublished; provide abstracts of significant items, rating them for lucidity, scope and completeness; and develop bibliographies.

Building products registration

Also under development by AIA is a building products registration service, which will provide registration of data on materials and products and convenient tabulation of properties of comparable items, after investigation of evidence provided by the manufacturers and acceptable laboratories. This information service will not guarantee the product, but will provide assurance that the advertising claims and reported test results are factually authentic. This service will also be self-supporting through fees from manufacturers to cover the cost of investigation and dissemination of the results.

American Society for Engineering Education elects officers

Dr. Frederick C. Lindvall, chairman of the division of engineering at the California Institute of Technology, is the new president of the American Society for Engineering Education.

Three vice-presidents were elected to serve with him: Dean Clarence L. Eckel of the college of engineering, University of Colorado; Professor Glenn Murphy, head of the department of theoretical and applied mechanics at Iowa State College; and Robert W. Van Houten, president of the Newark College of Engineering. John Gammell, director of graduate training at the Allis Chalmers Manufacturing Co., Milwaukee, was reelected treasurer. Both Dean Eckel and Professor Murphy are ACI members.

Bottling plant utilizes folded plate

The new Coca Cola Bottling Plant, Colorado Springs, Colo., is a complete shell structure; tilt-up walls support the shell roof, and the roof laterally supports the walls.

The roof structure is a four-element folded plate with a shell thickness of 4 in. The

spans are 40 ft and 60 ft and the bay spacing is 28 ft. Walls are of the usual tilt-up construction with cast-in-place column elements to joint the sections.

Carlisle Guy was building architect; Ketchum and Konkel, Denver, were structural engineers, and Baxter Construction Co. was the contractor.

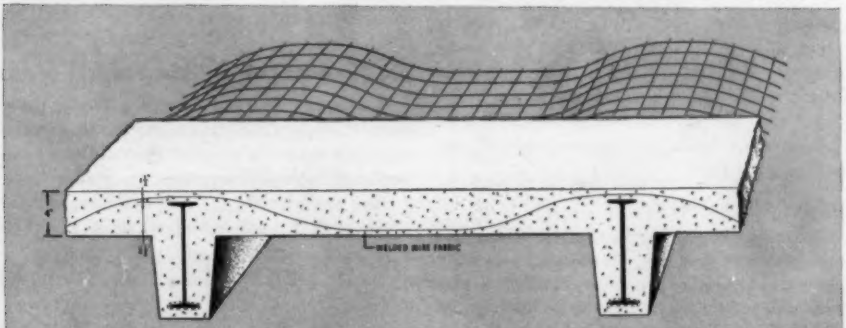
American Welded Wire Fabric in

Jutting its massive rentable floor space up 45 stories into the New York sky line, the Socony Mobil Building is full of innovations. Its high-voltage wiring system is one example. Its operatorless, self-service elevators another. But one place you will find no innovation is in the 1,300,000 sq. ft. of floor construction —and there is a good reason.

Short span beam-and-slab concrete floors were used and reinforced with American Welded Wire Fabric. This same reliable construction was used a quarter of a century ago in buildings in Rockefeller Center. It is still the best answer in modern buildings. It provides the structural strength, the light weight, the fireproofing and the ease of installation needed to build a good floor structure economically.

Many
Construction Materials

Welded Wire Fabric is one of many building materials manufactured by American Steel & Wire. Others include Super-Tens Wire and High Strength Strand for prestressed concrete applications. Write for complete technical information on these products.



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President

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Secretary-Treasurer

Date of membership: July 1, 1959

Above is shown the membership certificate now available to all interested members. (Student members become eligible for certificates when they advance to one of the higher membership grades.) The actual certificate is 10 x 12 in. on parchment paper suitable for framing.

Return this order form together with your remittance—to avoid the added expense of billing—and your certificate will be mailed to you. Please allow 90 days for preparation of the certificate.

Please accept my order for a membership certificate at the price of $2.00.

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(Print or type)

Signature_____

Stanton Walker receives ASTM Richart award

Stanton Walker, director of engineering, National Sand and Gravel Association and National Ready Mixed Concrete Association, Washington, D. C., received the Frank E. Richart Award during the 60th annual meeting of the American Society for Testing Materials. The award commemorates the late Frank E. Richart, a vice-president and honorary member of the society, and is made no oftener than every 3 years to encourage research and standardization and to recognize notable contributions in the field of concrete and concrete aggregates.

After his graduation from the University of Illinois in 1917, Mr. Walker became a research engineer for the Portland Cement Association. He has been director of engineering, NSGA, since 1926, and since 1930 has also been director of engineering of the National Ready Mixed Concrete Association. He is also director of the NSGA Research Foundation and of the NRMCA Research Laboratory both at the University of Maryland.

A past-president of ACI, Mr. Walker has been a member of ASTM since 1920 and is also a member of ASCE, the Highway Research Board, and AIME.

Building Fund Contributions

as of July 15, 1957

Cement producers	$ 69,512.00
Reinforcing steel industry	40,000.00
Engineers and architects in private practice	34,176.22
Contractors	18,445.00
Ready-mixed concrete and aggregate industry	10,983.87
Admixtures manufacturers	7,258.93
Membership at large	4,940.49
Concrete machinery and specialty products	2,585.00
Concrete products manufacturers	2,520.00
Eastern Canada	1,496.90
Special contributions	1,209.08
Total	$193,127.49
Pledges in hand	7,300.00
Grand Total	$200,427.49

See Seattle in November

The metropolis of the northwest will welcome ACI members, their families, and guests to the ACI Regional Meeting Nov. 5-6.

Technical sessions will precede the inspection trips arranged by R. H. Lochow and his assistants on the local planning committee. Scenic port city and aircraft manufacturing center, Seattle offers the visitor a tempting climate —average summer temperature is about 65 F; winter temperatures average 45. Convention headquarters will be the Benjamin Franklin Hotel; inquiries regarding accommodations may be addressed directly to the hotel.

More details in the September NEWS LETTER. Remember to save the dates:

November 5-6, 1957

ACI REGIONAL MEETING

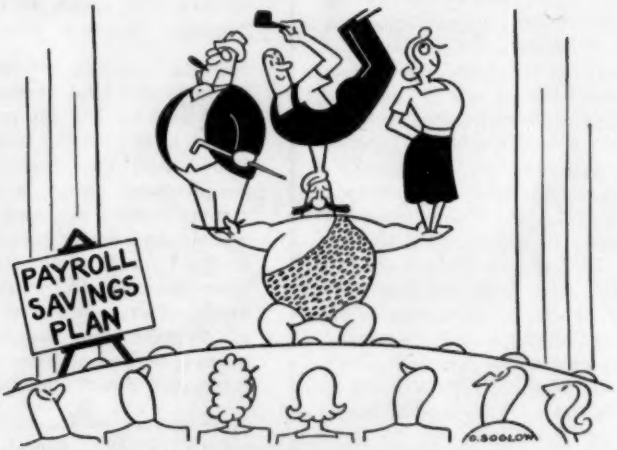
Benjamin Franklin Hotel

Seattle, Washington

German concrete group to meet in Berlin

Deutscher Beton-Verein (German Concrete Association) has announced the program for its "Deutscher Betontag 1957" to be held in Berlin, Sept. 24-26, 1957 in connection with the Berlin International Building Exhibition. Among ACI members scheduled to participate in the technical sessions are: Hubert Rüsch, Munich; Walter Nakonz, Düsseldorf; Kurt Walz, Düsseldorf; and Wolfgang Zerna, Hannover.

"Got A Load On Your Mind? I Haven't"



Nothing gives employees a more luxurious, satisfied feeling than that of financial security. And nothing is easier for them to achieve when you provide the convenience of automatic Payroll Savings Plan.

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Moreover, when you install the Payroll Savings Plan in your company, you promote not only the security of your personnel but the security of your company and your country. Over forty million Americans already have over 40 billion dollars invested in United States Savings Bonds—a tremendous backlog of purchasing power for the future.

EASY TO INSTALL

If your company does not now have a Payroll Savings Plan, or if employee participation is less than 50%, a letter to: Savings Bonds Division, U. S. Treasury Department, Washington, D. C., will bring prompt assistance from your State Director. He will provide application cards, promotional material, and as much personal help as you need.

United States Savings Bonds

Who's Who This Month

ACI Committee 208, Bond Stress

"Proposed Test Procedure to Determine Relative Bond Value of Reinforcing Bars" prepared by ACI Committee 208 appears on p. 89 of this month's JOURNAL. Chairman of this committee is C. P. Siess, University of Illinois, Urbana. Committee members are: H. J. Gilkey, Iowa State College, Ames; Arthur P. Clark, consultant, Washington, D. C.; Phil M. Ferguson, University of Texas, Austin; W. H. Jacobs, Rail Steel Bar Association, Chicago; R. C. Reese, consulting engineer, Toledo; David Watstein, National Bureau of Standards, Washington, D. C.; A. C. Weber, Laclede Steel Co., St. Louis; and C. A. Willson, American Iron and Steel Institute, New York. The report is being published with view to its consideration for adoption as an Institute standard. Since the preparation of this report S. J. Chamberlin, Iowa State College, Ames, has succeeded H. J. Gilkey.

Paul F. Rice

ACI technical director, Paul F. Rice presents "Design of Concrete Floors on Ground for Warehouse Loadings" appearing on p. 105. Mr. Rice studied at North Dakota State College, Illinois Institute of Technology, Massachusetts Institute of Technology (where he received his MS in civil engineering), and at the University of Michigan.

His professional experience includes structural detailing and shop inspection of fabricated steel, field inspection and laboratory testing of concrete, structural design, and estimating. He served 39 months during World War II in a U. S. Army aviation engineer battalion with responsibilities for paving, building construction, labor and supply procurement, and earth moving. Mr. Rice was with the Portland Cement Association for 5 years as a structural field engineer before coming to ACI in October 1954.

He is a member of ASCE, Society of American Military Engineers, Engineering Society of Detroit, Michigan Society of Professional Engineers, and is ACI representative to the Construction Specification Institute.

John J. Driskell

"Multispan Prestressed Folded Plate Roof for Langendorf Bakery" appearing on p. 115 is contributed by John J. Driskell, consulting structural engineer, San Gabriel, Calif.

A civil engineering graduate of Ohio State University in 1937, Mr. Driskell worked for 3 years with the American Bridge Co., then went to Panama for 2 years' work on lock protection projects. Moving to Los Angeles in 1941, he was structural engineer with the Corps of Engineers for 5 years, engaged in design of dams, flood control works, and bridges. Following work with several architectural firms, he joined the Portland Cement Association as field engineer, and then in 1949 he opened his own office where he has been engaged in the structural design of a wide variety of buildings. He has pioneered development of folded plate concrete roofs and the applications of prestressing techniques in Southern California.

Mr. Driskell is a member of ASCE, Structural Engineers Association of Southern California, and Tau Beta Pi; he has been a member of ACI since 1952.

L. Dean Sanders and W. J. Smothers

The combined efforts of L. Dean Sanders and W. J. Smothers have resulted in the paper entitled "Effect of Tobermorite on the Mechanical Strength of Autoclaved Portland Cement-Silica Mixtures" appearing on p. 127.

L. Dean Sanders received his MS degree in ceramic engineering from Ohio State University in 1955. Upon graduation he became a research engineer at the Ohio Brass Co., Barberton, Ohio. At present he is assigned to project for development of high strength cements to be used in the production of suspension insulators.

W. J. Smothers, a ceramic engineering graduate of Missouri School of Mines and Metallurgy in 1940, received his PhD at the University of Missouri in 1944. While still an undergraduate, he worked as cement chemist for the Missouri Portland Cement Co., St. Louis. He studied clays as additives to cement for use in mortars for a bachelor's thesis, later did research on polarographic determination of alkalies in portland cement. Mr. Smothers is now director of research at the ceramic research laboratory of the Ohio Brass Co.

Gene M. Nordby and William J. Venuti

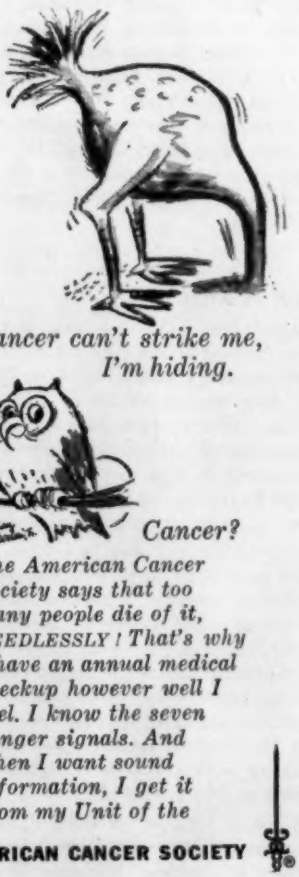
"Fatigue and Static Tests of Steel Strand Prestressed Beams of Expanded Shale Concrete and Conventional Concrete," p. 141, is the contribution of Gene M. Nordby of the National Science Foundation and William J. Venuti of San Jose State College.

Mr. Nordby graduated from Oregon State College in 1948 and received his MS and PhD from the University of Minnesota in 1949 and 1955 respectively. During 1950 he worked as a structural engineer for a firm of consulting engineers in Minneapolis, then became a member of the faculty of the University of Colorado from 1950 to 1956. During this period he was active in concrete research, studying all aspects of prestressed concrete, and the behavior of welded wire fabric in lightweight-aggregate concrete. In 1956 he became associate professor of structural engineering at Purdue University where he continued his prestressed concrete research. He is now on leave from Purdue at the National Science Foundation where he is administering a national program in research and education in the engineering sciences. Although he now deals with research in all engineering fields, he maintains active interest in the promotion of concrete research through the foundation and other groups.

Mr. Nordby is a member of the Reinforced Concrete Research Council, research committees of both ACI and ASCE, and the joint ACI-ASCE committee studying prestressed concrete; he is chairman of ACI Committee 215, Fatigue of Concrete.

William J. Venuti received his BS and MS degrees at the University of Colorado and is now engaged in graduate study at Stanford University. His professional experience includes design and stress analysis of concrete arch dams and irrigation canal structures with U. S. Bureau of Reclamation; concrete inspection with the USBR; and also engineering research on use of expanded shale aggregate in prestressed concrete for University of Colorado Engineering Experiment Station.

Mr. Venuti was an instructor in civil engineering at the University of Colorado from 1952 to 1955, and now is assistant professor of engineering at San Jose State College, San Jose, Calif. He also serves as senior design engineer for the Food Machinery and Chemical Corp., San Jose. He is affiliated with ASCE and the American Society for Engineering Education.


Cancer can't strike me,
I'm hiding.
Cancer?
The American Cancer Society says that too many people die of it, NEEDLESSLY! That's why I have an annual medical checkup however well I feel. I know the seven danger signals. And when I want sound information, I get it from my Unit of the
AMERICAN CANCER SOCIETY

Membership in the American Concrete Institute

To facilitate prospective members in joining the Institute, membership application forms are provided. Present Members may aid by bringing these forms to the attention of those who may profit from membership advantages. The grades of membership are described overleaf.

All who have an interest in concrete are eligible for membership.

Members have at hand in Institute publications the most complete fund of knowledge on concrete. The ACI JOURNAL provides them with the latest information and ACI special publications provide them with the complete picture of specific problems. Through conventions, and regional and area meetings they are afforded the opportunity of meeting those whose experiences provide the new information, and of exchanging ideas with them.

Opportunity for service is present in technical committee activity, in contributions, or only comments, to the ACI JOURNAL, or in reviewing technical publications for material of interest to the membership.

ACI's world-wide membership is growing in extent and participation—traveling a common road toward better, more economical and durable concrete structures. ACI provides a common ground in the search for and use of new "working tools" in concrete design, manufacture, and erection—and its interpretation.

(cut here)

Board of Direction, American Concrete Institute
P. O. Box 4754, Redford Station
Detroit 19, Michigan

Date_____

Individual Members (U. S. and Possessions, Canada, Mexico, Central America, and West Indies)	$15.00
Individual Members (All other foreign countries)	12.00
Corporation Members	50.00
Contributing Members	100.00
Junior Members—nonvoting (under 28)	7.50
Student Members—nonvoting (under 28)	5.00

(Subject to stipulations of Bylaws—Article I on reverse side. Bylaws on request.)

Of the annual dues, $12.00 is for the JOURNAL of the American Concrete Institute (except that dues for Junior and Student Members apply in full for the JOURNAL).

The undersigned hereby applies for_____**membership**
(Individual, Corporation, Contributing, Junior, Student)

in the American Concrete Institute. Proposed by_____

For Corporation Membership, ACI representative will be_____

_____ _____
(Date of graduation if Student) (Name, if Corporation)

Signature_____

For our records, please complete both sides of application.

EXCERPTS FROM BYLAWS: ARTICLE I—MEMBERS

Section 1. This Institute shall consist of Members, Corporation Members, Contributing Members, Junior Members, Student Members, and Honorary Members interested in furthering the Institute's objects as set forth in its Charter.

Sec. 2. A Member shall be an individual.

A Corporation Member shall be a firm, corporation, society, agency of government or other organization. A Corporation Member shall name one individual as its representative who will enjoy all membership rights and privileges.

A Contributing Member shall be an individual, firm, corporation, society, agency of government, or other organization wishing to give larger support to Institute activities through the payment of larger dues. Contributing Members, other than individuals, shall name representatives as do Corporation Members.

A Junior Member shall be a person less than 28 years old.

A Student Member shall be less than 28 years old and a student in residence at a recognized technical or engineering school.

An Honorary Member shall be a person of eminence in the field of the Institute's interest, or one who has performed extraordinarily meritorious service to the Institute.

Sec. 3. All classes of Members except Honorary Members and Student Members, shall be proposed by at least one Member of the Institute and elected by a two-thirds vote of the Board of Direction. An Honorary Member shall be elected by unanimous vote of the Board of Direction. A Student Member shall be proposed by a member of his school's faculty, who need not be a Member of the Institute, and elected by a two-thirds vote of the Board of Direction.

Sec. 4. All Members shall have all rights and privileges of membership as determined by the Board of Direction except that a Junior or Student Member shall not vote nor hold office. The status of a Student Member shall change automatically to that of Junior Member on the first anniversary of his membership succeeding the date on which he ceases to be a student in residence. The status of a Junior Member shall be changed to that of Member on the first anniversary of his membership after he becomes 28 years of age.

Sec. 5. Applications for and resignations from Membership and requests for change of representatives of Corporation or Contributing Memberships shall be presented in writing to the Secretary-Treasurer. Resignations may be accepted only from Members whose dues are not more than 60 days in arrears, except by special action of the Board of Direction.

(cut here)

Name _____
Please Print

Date of Birth _____
Month Day Year

Title or Position _____

Name of Firm or Organization _____

☐ **Business Address** _____

☐ **Resident Address** _____
(Please check address to which you wish mail and publications sent)

Nature of Firm's Business _____

The ACI Membership Directory will be sent—as available—only on request.
Check here if you wish to receive the latest edition. ☐

Honor Roll

February 1—June 30, 1957

First and second place remain the same this month with G. B. Southworth in first place with 13 credits and Arthur N. L. Chiu in second with 7 credits. Martin J. Gutzwiller and H. C. Pfannkucke are now tied in third place with 6 credits each.

Have you made your contribution to the growth of ACI? If you know at least one person who is active in the concrete field introduce him to the benefits of a membership in ACI.

G. B. Southworth	13
Arthur N. L. Chiu	7
Martin J. Gutzwiller	6
H. C. Pfannkuche	6
Phil M. Ferguson	5
Walter H. Price	5
Luis Perez Cid	4
Jaime de las Casas	4
Louis A. Gottheil	4
Henry L. Kennedy	4
L. M. Legatski	4
James A. McCarthy	4
S. E. Acker	3
James C. Amrhein	3
Nicandro Barboza	3
R. F. Blanks	3
W. S. Cottingham	3
C. A. Engman	3
Ambrosio R. Flores	3
Myle J. Holley, Jr.	3
J. E. Jellick	3
Franklin B. Johnson	3
I. Martin	3
Martin Schulz	3
Howard Simpson	3
H. T. Williams	3
John T. Young	3
Miles N. Clair	2½
W. H. Armstrong	2
Geo. F. Bishop	2
Jose L. Capacete	2
James Chinn	2
Dale Cobb	2
Roger D. de Cossio	2
E. I. Fiesenheiser	2
Kenneth H. Gedney	2
Roger E. Harlepp	2
N. L. Hinkson	2
Samuel Hobbs	2
M. W. Huggins	2
Leo Liberthson	2
Robert H. Lochow	2
Patrick McNally	2
Frederick T. Mavis	2
Izuo Miyashita	2
George H. Nelson	2
Richard A. Parmelee	2
Abdur Rahman S. Rasul	2
Theodore O. Reyhner	2
Sabri Sami	2
Harry Saxe	2
William C. Schwenger	2
Joaquin Spinel L.	2
E. W. Thorson	2
Ellis S. Vieser	2

New Members

The Board of Direction approved 68 Individua applications, 1 Corporation, 18 Juniors, and 17 Students making a total of 104 new members. Considering losses due to resignations, deaths and nonpayment of dues, the total membership on July 1, 1957 was 8873.

Individual

ANDERSON, BERT THOMAS, Glenview, Ill. (Arch., Consoer & Morgan, Archs.)

ANTILL, J. M., Rose Bay, NSW, Australia (Mng. Dir. & Chf. Engr., Stresscrete Constructions Pty Ltd.)

ARAI, KEIZO, Osaka, Japan (Vice-Pres., P. C. Bridge Co.)

ARROYO R., ALBERTO, Caracas, Venezuela (Ingeniero Calculo Estructural y Revision de Construcciones, Compania Anonima Venezolana de Cemento)

BEAUCHAMP, EDWARD JOSEPH, Toronto, Ont., Canada (Bldg. Engr., City of Toronto, Bldgs. Dept.)

BROWN, VERNON B., Miami, Fla. (Owner, Brown Seels & Assocs.)

BUSCK, THEODOR H., Whangarei, New Zealand (Mng. Dir., Busck Concrete Ltd.)

CAREY, JAMES W., Seattle, Wash. (Cons. Engr., Carey & Kramer, Cons. Engrs.)

CHRISTIE, NORMAN J. R., Cooma North, NSW, Australia (Constr. Engr., Supv., Admin., Snowy Mountains Hydro-Electric Authority)

CHUTIKUN, PAIRAT, Bangkok, Thailand (C. E., Tech. Div., Royal Fine Art Dept.)

CONACHER, J. E., London, SW, England (Chf. Engr., Blaw-Knox Ltd.)

CONDE S., SAMUEL S., Ciudad Trujillo, D. S. D., Rep. Dom.

COSTA, ARTURO, Hato Rey, Puerto Rico (Proj. Engr., Designing, Inspection, Las Monjas Development Corp.)

COX, DEMMIE H., Houston, Texas (Vice-Pres., Houston Mgr., Texas Gulf Constr. Co., Inc.)

DAIGLE, H. D., Montreal, Que., Canada (Sales Engr., Constr. Specialties, Dewey & Almy Chemical Co. of Canada, Ltd.)

DAVIDSON, R. D., Toronto, Ont., Canada (Exec-Sec., National Concrete Products Association)

DAVIES, CYRIL DERRICK, Stourbridge, Worcestershire, England (Civil Engr. Agent, E. E. Jeavons & Co., Ltd.)

EDLEY, DAVE, Hoboken, N. J. (Chief, Engrg. Inspection, United States Testing Co., Inc.)

FERNANDEZ R., CARLOS, Habana, Cuba (Ingeniero Auxiliar de Correteras, Ministerio de Obras Publicas)

GARDNER, WILLIAM S., Ciudad Trujillo, Dominican Republic (Cons. Engr., Highways & Airports)

GRASSI, ADRIANO, Caracas, Venezuela

GUTIERREZ, RAFAEL, Santiago de Cuba, Cuba (C.E.)

HALL, JOHN F., New York, N. Y. (Mgr., Tech. Dept., Keystone Cement Co.)

HENNESSEY, D. E., London W1, England (Cons. Engr., Civil & Struct., Messrs. Bolton, Hennessey & Partners)

HERNANDEZ M., FEDERICO, Mexico, D. F., Mexico (C.E., Bloques y Ladrillos, SA)

HERSUM, LEROY M., Boston, Mass. (Cons. Engr.)

HOLLINGSWORTH, LESTER P., Lake Success, N. Y. (Struct. Engr., Byrne Assocs.)

JOE, ALVIN K., Berkeley, Calif. (Sr. C.E., East Bay Municipal Utility District)

JONAS, HYMEN, Milwaukee, Wis. (Stud. Civil Engrg.)

JOSECK, HARRY C., Pittsburgh, Pa. (Asst. Chf. Draftsman, Pittsburgh Coke & Chemical Co.)

KARNI, JOSEPH, Haifa, Israel (Sr. Lecturer, Technion, Israel Institute of Technology)
KELSO, ALVIN N., Galveston, Texas (Constr. Engr., Gen. Constr.)
KILROY, STANLEY ALAN ROBERT, Flemington, Victoria, Australia (Engr.)
KINDSVATER, E. F., Carlsbad, Mexico (Pres. & Gen. Mgr., Farm Chemical Resources Development Corp.)
KLEIN, LEONARD L., Detroit, Mich. (Engr., Gen. Engrg., Superior Products Co.)
LAU, CHEW LUN, Honolulu, Hawaii
LAUT, RICHARD W., Cleveland Heights, Ohio (Sales & Field Engrg., The Master Builders Co.)
LEE, HARRY, Ann Arbor, Mich. (Engr., McNamee, Porter & Seeley)
LEHMANN, KENNETH F., Hinsdale, Ill. (Chf. Struct. Engr., Battey & Childs)
MADSEN, RICHARD A., Minneapolis, Minn. (Vice-Pres., Twin City Readymix Concrete Co.)
MAGINN, WILLIAM F., Masena, N. Y. (Pres., St. Lawrence Transit Mix Corp.)
MINASI, ANTONIO, San Salvador, El Salvador (C.E.)
MATHEY, ROBERT G., Hyattsville, Md. (Struct. Engr. in R/C, NBS)
MAWER, CHARLES J., Gisborne, New Zealand (Clerk of Works, Ministry of Works)
NUGENT, W. S., Jackson, Mich. (Vice-Pres., Sales Dir., Consolidated Cement Corp.)
PORTNOW, IRVING, Flushing, N. Y. (Proj. Engr. & Gen. Supt., University Towers)
RADA A., VICTOR MANUEL, Caracas, Venezuela (Insp. & Struct. Engr., Fibrocemento, S. A.)
RAFANELLI, LOUIS R., West Richfield, Ohio (Field Engr., Roediger Constr. Co.)
RICHARDSON, PETER CAMERON, Manchester, England (Chemist, FED (Great Britain) Ltd.)
ROBERTS, JOHN T., Miami, Fla. (Vice-Pres. & Gen. Mgr., H. C. Nutting Co. of Florida)
ROBERT, WALTER F., Santa Fe, New Mexico (Asst. Proj. Engr., N. M. State Hwy. Dept.)
ROTH, GREGORIO, Jamaica, N. Y. (Struct. Engr., Patterson, Emerson, Comstock, Inc.)
RUSSELL, LESLIE W., Vancouver, B. C., Canada (Gen. Mgr., Globe Chemicals Ltd.)
SEMANDS, EVERETT D., Houston, Texas (Struct. Engr., Austin Co.)
TENCHAVEZ, PASTER B., Manila, Philippines (Prof. C. E., Struct. Design, Atlantic, Gulf & Pacific Co. of Manila)
TENNIS, EDGAR W., Springfield, Del. Co., Pa. (Struct. Engr., Allabach, Rennis, Inc.)
TORRES DE ALBA, ENRIQUE, Mexico, D. F., Mexico (Director de Servicios Tecnicos, Division de Alambre de Acero, "La Consolidada S. A.)
TOSTA, HECTOR F., Caracas, Venezuela (C.E., Servicio de Ingenieria Militar)
VAN EPPS, ROBERT J., Chicago, Ill. (Struct. Engr., Tech. Writing & Development, PCA)
VILLANI, V. A., c/o Vicei Ltd., Azisiyah, Iraq
WASELL, R. V., Charlotte, N. C. (Cons. Engr. in Struct. & Civil Engrg.)
WELTER, RICHARD CHARLES, Indianapolis, Ind. (Western Electric Co.)
WHEELER, RUSSELL K., Pensacola, Fla. (Owner, Pensacola Testing Labs.)
WILSON, GEORGE W., Ft. Lauderdale, Fla. (R. H. Wright & Son)
ZEITMAN, M. KAMEL, Giza, Egypt (Dir. of Arch. Section, Building Research & Training Center)
ZELLER, M. S., Miami, Fla. (Bldg. Inspector, Dade County Planning, Zoning & Bldg. Dept.)

Corporation

TURBOTVILLE BLOCK CO., INC., Turbotville, Pa. (Allen G. Stamm, Pres.)

Junior

ALMA JOSE, JOSE P., Ann Arbor, Mich. (Struct. Engr., Giffels & Vallet, Inc.)
AZAM, M. H., Dacca, East Pakistan (Lecturer in Ahsanullah Engrg. College, Civil Engrg. Dept., Gov't of East Pakistan)
BASSIM, PEDRO MOHSEN, Edo Anzoategui, Venezuela (C.E., Mene Grande Oil Co.)
COSTINETT, JOHN HENRY, JR., Hyattsville, Md. (Engrg. Draftsman, Foundation & Matls. Branch, Corps of Engrs.)

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Sample remains intact. Small amount of water used in test permits using same sample for slump and compression tests. Universal acceptance — America, Europe and Asia. Specific gravity and moisture determinations quickly made using chart.

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EVANS, IAN A. N., Baghdad, Iraq (Insp. Engr., Res. Engrs. Office, Derbendi Khan Project, Harza Engrg. Co.)
GUPTA, CHANDAN KUMAR, London W, England (Designer & Detailer, C. J. Pell & Partners)
HANSEN, EINAR BLOCH, St. Louis, Mo. (Struct. Engr., Sverdrup & Parcel, Inc.)
HOLT, LAURENCE G., Woodbury, N. J. (Bridge Engr., Brown & Blauvelt)
LEONARDSON, KENNETH R., Elmhurst, Ill. (Jr. Engr., Harza Engrg. Co.)
LIN, KUANG-MING, Auburn, Ala. (Grad. Stud., Alabama Polytechnic Inst., Research Asst.)
MARSHALL, DONALD WILLIAM, Houston, Texas (Struct. Designer, Walter Kidde Engrs., Inc.)
ROTONDO, PHILIP L., Avon, Conn.
RUDOLPH, RICHARD H., 2nd Lt., #04045084, New York, N. Y. (Supv. Const. Platoon, U. S. Army)
SALAZAR MORTON, DANIEL, Mexico D. F., Mexico (Constr. Supt., Constructora Progreso S. A.)
SMART, REX, Hobart, Tasmania, Australia (Constr. Supv., Stephenson & Turner, Archs. & Engrs.)
TSIUNG, DONALD, Brookline, Mass. (Designer, Cleverdon, Varney & Pike, Cons. Engrg. Co.)
WARVI, MARTIN MATTHEW, Lead, S. Dakota (Grad. Engr., Brady Engrg.)
WINSLETT, USHER T., Jr., Macon, Ga. (C.E., Law-Barrow-Agee Labs.)

Student

AGUERREVERE, FRANCISCO, Caracas, Venezuela (Univ. Catolica Andres Bello)
ARCAY PAZ, MANUEL, Caracas, Venezuela (Univ. Catolica Andres Bello)
ARECHABALETA, INAKI, Caracas, Venezuela (Univ. Catolica Andres Bello)
AYALA L., CARLOS, Caracas, Venezuela (Univ. Catolica Andres Bello)
BELLO, PEDRO PAUL, Caracas, Venezuela (Univ. Catolica Andres Bello)
BOLINAGA IBARRONDO, JUAN JOSE, Caracas, Venezuela (Univ. Catolica Andres Bello)
DOMINGUEZ MORALES, ENRIQUE, Maracaibo, Venezuela (Univ. Nacional del Zulia)
DURAN RODRIGUEZ, JOSE, Caracas, Venezuela (Univ. Catolica Andres Bello)
ERASO, PABLO CEBALLOS, Caracas, Venezuela (Univ. Catolica Andres Bello)
FREEMAN, SIGMUND ARNOLD, San Francisco, Calif. (Univ. of Calif.)
LANDAES, JESUS MARIA, Caracas, Venezuela (Univ. Catolica Andres Bello)
MARTURET, JUAN AZPURUA, Caracas, Venezuela (Univ. Catolica Andres Bello)
MITCHELL, NEAL B., Jr., Providence, R. I. (Brown Univ.)
PATINO, JOSE NILO, Fort Wayne, Ind.
SOLORZANO, TOMAS ANT, Caracas, Venezuela (Univ. Catolica Andres Bello)
VASQUEZ M., JUAN, Caracas, Venezuela (Univ. Catolica Andres Bello)
YAO, JAMES T. P., Champaign, Ill. (Univ. of Ill.)

International road congress

The International Concrete Road Congress to be held in Rome, Oct. 16-19, 1957, will have as its theme "Recent Technical Progress and Achievements in the Construction of Concrete Roads and Airfield Runways."

Official languages of the congress will be English, French, German, and Italian. There will be simultaneous translation into all four languages during the proceedings.

The meeting is under the auspices of the cement manufacturers associations of Belgium, Denmark, Finland, France, Germany, Great Britain, Italy, the Netherlands, Norway, Sweden, Switzerland, and "Cembureau" (the Cement Statistical and Technical Association of Malmö).

Frederick T. Mavis becomes dean

Frederick T. Mavis has been made dean of the University of Maryland's College of Engineering. He had been head of the civil engineering department at Carnegie Institute of Technology since 1944. Before joining the Carnegie faculty he was professor and head of the civil engineering department at Pennsylvania State College.

LOOKING AHEAD

September 8-13, 1957—American Chemical Society, National Meeting, New York, N. Y.

September 9-13, 1957—Instrument Society of America, 12th Annual Instrument-Automation Conference and Exhibit, Auditorium, Cleveland, Ohio.

September 23-25, 1957—Standards Engineers Society, Sixth Annual Meeting, Hotel Commodore, New York, N. Y.

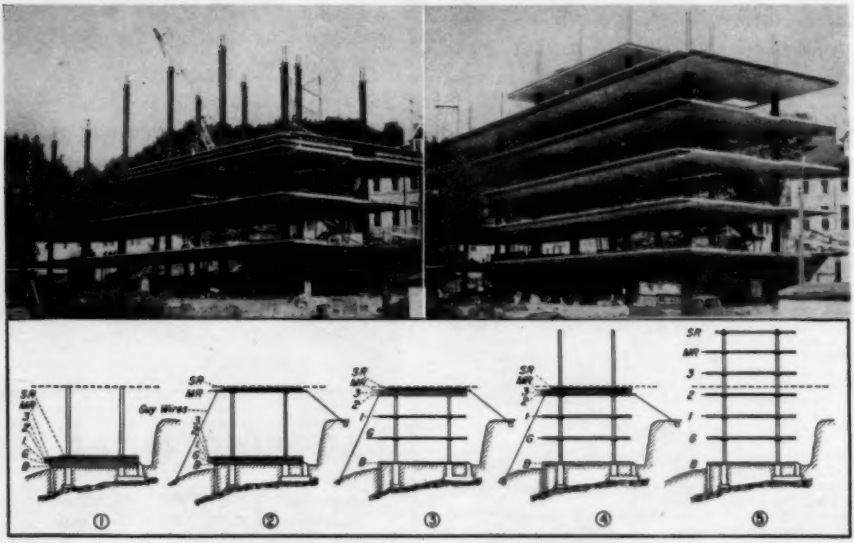
October 9-11, 1957—National Slag Association, 40th Annual Meeting, Plaza Hotel, Washington

October 14-18, 1957—American Society of Civil Engineers, National Convention, Statler Hotel, New York, N. Y.

October 18-19, 1957—National Society of Professional Engineers, Fall Meeting, Grand Pacific Hotel and Provident Insurance Building, Bismarck, N. D.

November 5-6, 1957—ACI Regional Meeting, Benjamin Franklin Hotel, Seattle, Wash.

November 13-15, 1957—American Standards Association, Annual Meeting and National Conference on Standards, St. Francis Hotel, San Francisco, Calif.



Why They Used PLASTIMENT in this 6-Story Prestressed Lift-Slab

Combining two recent construction and design techniques – lift slab and prestressing – the Litchfield County Hospital at Winsted, Connecticut is an outstanding example of advanced engineering.

Concrete of uniform high quality was essential to the success of the project. According to the engineer:

"Plastiment was added to the concrete to reduce the amount of water and shrinkage and to improve workability."

Plastiment has been selected by engineers for many outstanding projects because of its proved ability to increase concrete uniformity and strength, improve placeability at low slump, and reduce cracking.

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Bovay announces merger

Harry Elmo Bovay, Jr. of the H. E. Bovay, Jr. consulting engineering firm of Houston, announces the merger of this company with the Reg F. Taylor firm. The new company will continue under the name of H. E. Bovay. Other partners in the Bovay firm are Bennett W. Burns, C. A. Lawler, Richard D. Robertson, and W. N. Pauley.

Bloodgood honored

Grant Bloodgood, chief construction engineer, Denver, was among six U. S. Bureau of Reclamation employees recently honored with the Distinguished Service Award, the highest honor the Department of the Interior can bestow. Mr. Bloodgood has been with the bureau since 1920; he joined ACI in 1948.

New BRI governors appointed

Two ACI members, F. E. Fahy, technical adviser, research department, Bethlehem Steel Co., Bethlehem, Pa., and Minoru Yamasaki of Yamasaki, Leinweber & Associates, Royal Oak, Mich., were recently appointed to serve 3-year terms as members of the Building Research Institute's board of governors.

Harry Plummer, Washington, director, engineering and technology, Structural Clay Products Institute, retired from the board this year and was awarded a certificate of appreciation for his part in progress of the Building Research Institute during the past 3 years.

American Concrete Agricultural Pipe Association officers

Progress in all fields of concrete agricultural pipe was reported at the 7th annual convention of the American Concrete Agricultural Pipe Association in Denver recently. Representatives of the drain tile, irrigation pipe, and low-head pressure pipe industries met with engineers from the Soil Conservation Service and the U. S. Bureau of Reclamation to consolidate gains and to plan for the coming months. New ASTM specifications for concrete drain tile are in the making and may be ready for adoption this fall.

Officers re-elected for the coming year are: president, J. J. Seale, Concrete Conduit Co., Colton, Calif.; vice-presidents, Bruce N. Spencer, Jr., Gifford-Hill-Western, Inc., Lubbock, Tex., and Homer Peterson, Hutchinson, Minn.; secretary, Glenn Harriman, Indiana Lock Joint Co., Lafayette, Ind.; and treasurer, William B. Freeman, Lock Joint Pipe Co., Denver.

Tools, Materials, Services

Under this heading note is made of producer literature and products of presumed technical interest to ACI users of tools, equipment, materials, accessories, and special services.

Engineering examination book

An enlarged and revised book covering examinations given by the state of California for the registration of civil engineers and engineers in training is now available. For examinations given between 1940 and July 1949, book includes unofficial solutions of the problems by the author. Examinations from July 1949 to and including June 1956 include the problems only. All options for the civil engineers' examinations are included together with the engineer-in-training examinations for all branches of engineering. Containing 381 pp., book sells for $7.00.—August E. Waegermann, 2833 Webster St., San Francisco 23, Calif.

Water control system

The Forrer Co. has been appointed national distributor for the Ramsey Water Control System, an electronic device to control moisture in concrete mixes. System operates on the change of the electrical resistance of the mix as water is added; it measures total water content of mix, not just amount being added. Unit is guaranteed for a year. Controls are manufactured and serviced by Minneapolis Honeywell. —Forrer Co., 2225 N. Humboldt Ave., Milwaukee, Wis.

Coloring concrete

Folder containing color chips describes uses of Harcol Redi-Mixed, non-metallic dry mix for hardening and coloring concrete floor toppings. The dry powder is dusted on the fresh concrete surface and then final finishing is done. Manufacturer states color is chemical and sun resistant, and the treatment increases abrasion resistance.—L. Sonneborn Sons, Inc., Building Products Division, Dept. AA, 404 Fourth Ave, New York, N. Y.

Concrete drilling machine

The Eager Beaver will drill holes in reinforced concrete, brick, tile, stone, or any hard masonry material. The machine will drill either horizontally or vertically, is powered by 110 volt electric current. Drills holes up to 4 in. in diameter and up to 18 in. or more in depth at cutting rate of up to 2 in. per min.—Beaver Products, Inc., Somerset, Ky.

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AMERICAN CONCRETE INSTITUTE

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P.O. Box 4754, Redford Station Detroit 19, Mich.

Concrete cutter

New Felker concrete cutters, Models 257 and 367, feature increased spindle power through non-slip, high efficiency timing belt drive. A Vickers variable speed

hydraulic power drive to the rear wheels enables cutting on steep grades without slipping, according to manufacturer. Reverse is now provided for backing up under power.—Felker Manufacturing Co., Torrance, Calif.

Handling concrete products

"Effective Ideas for Handling Concrete Products," a 26-p. booklet, illustrates uses of industrial trucks and attachments. Handling of block, aggregate, pipe, pipe forms, and heavy panels is shown along with procedures for loading autoclaves and attachments for placing concrete.—Hyster Co., 2902 N. E. Clackamas St., Portland 8, Ore.

Cement distribution station

Bulletin G-4 describes cement distribution station of Olympic Portland Cement Co. at Harbor Island, Seattle. Text and photographs detail unloading cement from barge with Fuller-Kinyon remote control unloader, as well as handling of cement from silos by Airslide conveyors to bulk cement trucks and bag packers in the plant.—Fuller Co., Catasauqua, Pa.

Steel forms for precasting

Detail drawings and dimensions of steel forms are given in 16-p. Bulletin No. 200 describing Form-Crete products for precast and cast-in-place concrete. I-beams, bridge beams, square and octagonal pilings, bridge deck, and lintels are among the members for which standard forms are available. Also illustrated are forms for single- and double-T joists and double-T slab. Manufacturer reports 37 different castings can be made with the one double-T-joist form assembly. —Food Machinery and Chemical Corp., Florida Division, Lakeland, Fla.

Nonshrink mortar

A 4-p. folder illustrates the various uses of nonshrink Embeco mortar. Describes 12 important construction operations using the mortar and cites bulletins covering each particular application described.—The Master Builders Co., 7016 Euclid Ave., Cleveland 3, Ohio.

Plastic compound brochure

A brochure on three plastic compounds for joining, filling, cladding, and surfacing masonry materials, gives instructions for their use.—Permagile Corp. of America, 37-23 Thirty-third St., Long Island City 1, N. Y.

Prestressed tanks bulletin

Bulletin T-15 describes prestressed concrete tanks for processing plants, pulp and paper mills, cement plants, water storage and supply facilities, and water treatment, covering both design and construction data. Also includes condensed data on the use of prestressed concrete for containment vessels, pressure vessels, silos, and shell foundations.—The Preload Co., Inc., 211 E 37th St., New York 16, N. Y.

Rotary masonry drill

Catalog No. TD-2-57 describes complete line of Termite rotary masonry drills including prices and recommended sizes. Drills without the use of water. —Termite Drills, Inc., 99 N. Lotus Ave., Pasadena, Calif.

Water-reducing admixtures

Water-reducing admixtures for concrete now being produced by Marathon Corp. bear the trade name Maracon. These cement dispersants are based on fractionated lignosulfonates, available in the form of nonhygroscopic powders readily soluble in water and reportedly free of contaminants considered detrimental to concrete mixes.—Marathon Corp., Rothschild, Wis.

Rubber sewer pipe joints

A 20-p. booklet now available describes rubber gaskets for leak-proof coupling of concrete and vitrified clay sewer pipe. Booklet traces the story of the development of rubber joints and gives complete engineering and application data on Tylox rubber gaskets. Also contains illustrated case histories of installations in this country and abroad.—Hamilton Kent Manufacturing Co., 427 W. Grant St., Kent, Ohio

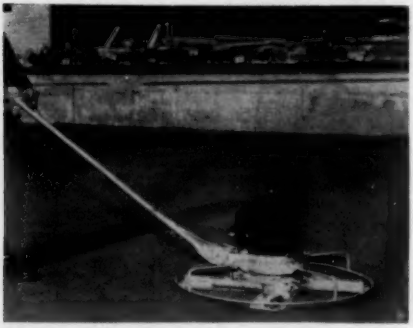
Shallow subsurface exploration without drilling

An instrument for shallow subsurface exploration without drilling, model MD-1 Refraction Seismograph, is used for determining depth to bedrock, the presence or absence of bedrock or solid material at any depth up to 50 ft, and identification of subsurface materials

without drilling. The instrument features direct reading, simple operation, and weighs 16 lb exclusive of sledge hammer source.—Geophysical Specialties Co., 4206 Longfellow Ave., Minneapolis, Minn.

Portable power trowel

Portable 34-in. power trowel features power blade adjustment; Master Powermatic Turn-a-Trowel allows the operator to change to any blade position between



float and finish without effort, manufacturer reports. Trowel measures only 14 in. from slab to top of engine, eliminating wobble and increasing stability and ease of operation.—Master Vibrator Co., 167 Stanley Ave., Dayton, Ohio

Conveyor for placing concrete

. A 4-page booklet (No. 157) gives specifications, photos and operating features of the Faircrete conveyor, designed especially for placing concrete. Mechanical features include automatic belt wiper, self cleaning foot pulley and triple ball bearing troughing idlers. Standard length is 40 ft and the conveyor is mounted on pneumatic tires to be towed behind car or truck.—Fairfield Engineering Co., Marion, Ohio

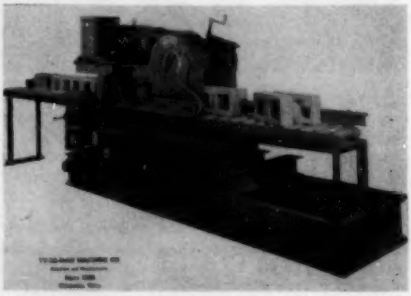
Volume measurement, moisture determination device

Model VM-500A Volumeter determines volumes of substances by air displacement. Either absolute volume or apparent volume may be measured. Designed for field or laboratory use, instrument is suitable for determining specific gravity, moisture content, density, and other physical properties of materials. —D. Ballauf Manufacturing Co., Inc., 619-21 H Street Northwest, Washington 1, D. C.

Saw for standard or odd-size blocks

Block saw produces two 4 x 8 x 16-in. blocks from one 8 x 8 x 16-in. unit or odd-size blocks of any height. The saw handles up to 600 blocks per hr with two unskilled workers loading and unloading.

Blocks are loaded on a roller conveyor and pushed through the cutting unit by two horizontal feed rollers.



Cutting unit consists of two diamond blades rotating in a vertical plane one above and one below the block. —Ty-Sa-Man Machine Co., 1043 White Ave., Knoxville Tenn.

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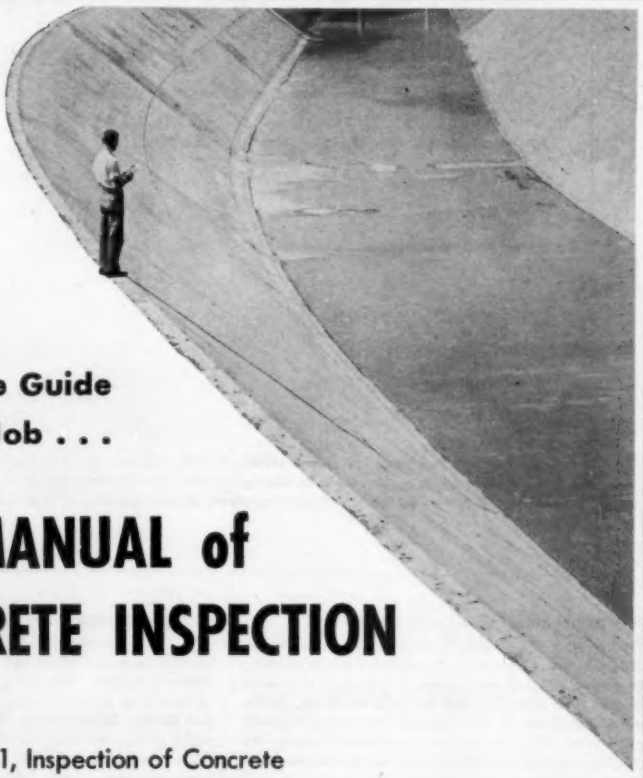

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Translation of Russian trade journal *Cement*

Complete English language translation of the Russian cement industry's trade journal *Cement* is now entering its second year of publication. Translation is done by specialists in the cement field and all tables, figures and illustrations in the original text are accurately reproduced. The journal, which generally contains ten to fifteen articles written by practicing Soviet engineers covers cement research, production equipment, and discusses specific problems encountered in various Russian factories. In addition there is a section devoted to reviews of foreign journals and surveys of the foreign industry. Annual subscription price for six translated issues is $60.00. Individual articles are $7.50. Tables of contents are provided free on request. —Consultants Bureau, Inc., 227 West 17th St., New York 11, N. Y.

IMPORTANT

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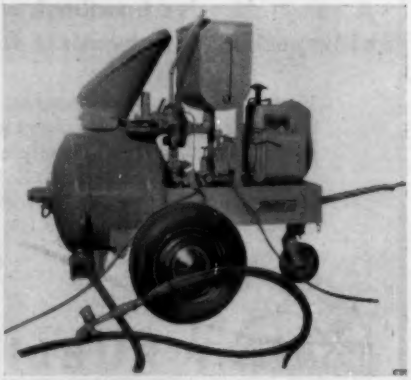
Multi-purpose masonry spray gun

Mobile pneumatic masonry spray gun designed for multiple-purpose, large and small-volume application is now available. The Sprayton Model 18 gun applies up to 3½ cu yd per hr, and will mix and spray stucco and plaster; apply cement coatings and build concrete walls; pump insulating roofing or floor topping; grout walls, machine bases or earthworks; and perform wet sandblast operations, according to manufacturer. Mounted on pneumatic-tired wheels, the gun has an



integral hopper and screen; a quick-change filler; a patented non-clogging outlet valve; a trigger-operated nozzle and pressurized seals in the mixing tank stuffing boxes.—Sprayton Equipment Co., 18055 James Couzens Highway, Detroit, Mich.

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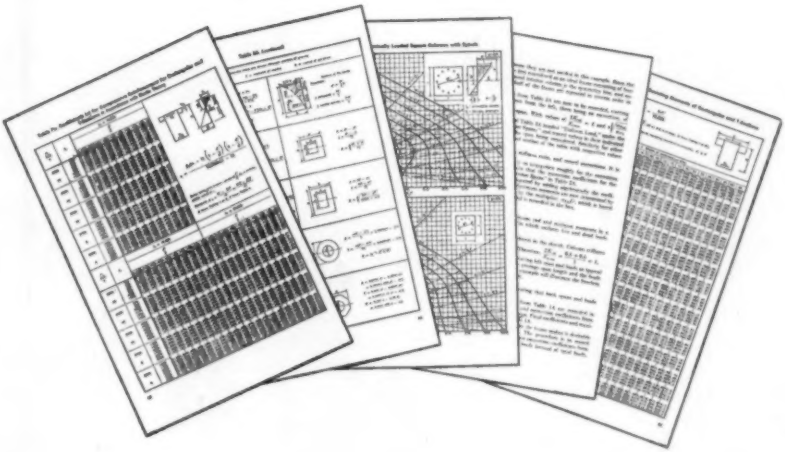

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